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A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 10, NO. 3 ▽ MAY, 1929

FRICTIONLESS CLAY

COHESIONLESS SAND

Subgrade Stability Depends on the Cohesion and Internal Friction of the Soil

U. S. GOVERNMENT PRINTING OFFICE: 1929

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INTERRELATIONSHIP OF LOAD, ROAD AND SUBGRADE

Present Status of Knowledge on the Subject with Suggestions for Applying this Knowledge in Practice

Reported by C. A. HOGENTOGLER, Senior Highway Engineer, and CHARLES TERZAGHI, Research Consultant, U. S. Bureau of Public Roads

An examination of existing data makes clear that attempts to explain pavement condition on the basis of subgrade soil types and the results of soil tests, or attempts to utilize in practice the information furnished by soil tests or by investigations of soil types must, of necessity, result in confusion unless there exists: First, a clear understanding of the subgrade properties which influence pavement behavior; second, a definite conception of basic pavement characteristics; and third, a knowledge of the significance of subgrade soil tests.

> THE discussion and conclusions which comprise this report are intended to illustrate the possibilities of utilizing soil and pavement data as a guide to the selection of the proper pavement type and the proper design of the type selected. It is also intended to point out how a knowledge of the primary subgrade soil characteristics considered with that of the different pavement properties may prevent the engineer from specifying one type of pavement when conditions will not permit of its adequate construction or operation; from increasing pavement thickness in an attempt to eliminate troubles not caused by lack of thickness; from attempting to cure structural defects by improving the wearing course; from attempting to prevent abrasive troubles by improving the structural resistance; from attempting to estimate the comparable service values of rigid and nonrigid pavements for conditions which clearly indicate the necessity for pavements furnishing high load distribution; from supplying only cohesion when friction also is required and vice versa; and from improving the subgrade to prevent troubles resulting from improper design, construction or maintenance of the pavement. In all cases, the underlying causes of trouble should be determined and the remedy should be selected with regard to the causes.

One should know the subgrade properties which are due primarily to the raw constituents of the soils; which are dependent primarily upon soil structure; and those which are due to field conditions under which soils exist.

One should be able to distinguish between defective pavement conditions which are caused entirely by the character of subgrade support, those which are caused only in part by the character of the support, and those which occur independently of subgrade conditions. One should also have a conception of the individual conditions of subgrade support which produce particular conditions in the pavement; the relative influence which similar subgrade conditions exert upon pavements different in type; the influence which pavement design and construction exert upon pavement behavior; and the pavement conditions which represent ultimate failure.

One should know the soil characteristics disclosed by a particular group of laboratory test results; the individual raw constituents which cause the soil to exhibit particular physical properties; the relative influence which the arbitrary test procedure and the physical characteristics of the raw constituents of soils exert upon the results furnished by laboratory tests; and finally, the extent to which the physical manifestations disclosed by laboratory tests serve to explain the behavior of soils when subjected to field conditions.

The information furnished by subgrade investigations in their present status suggests the logical courses to follow in the performance of new research and the benefits furnished by subgrade and pavement data when used in practice.

In presenting their material, the authors discuss first the results of completed research in conjunction with the information furnished by the highway engineer's every-day experience. Then follows a discussion of the theory which explains, at least in part, the behavior of the different subgrades under load. Finally, comes an application of the results of research to the design of roads.

The report is intended to disclose the relative load distribution furnished by several pavement types and the influence exerted by pavement characteristics and subgrade conditions upon pavement behavior; to inform one of the variables which cause certain particular conditions of subgrade support; to suggest a grouping of subgrades according to performance; to present a theory to explain the stability of subgrades, foundations, and pavements of certain types; to describe the functions of the various pavement variables and point out how to employ the different pavement variables properly with respect to subgrade conditions.

LOAD DISTRIBUTION BY DIFFERENT TYPES OF PAVEMENTS DISCUSSED

For the purpose of this discussion, road surfaces are divided into those which offer very little or no resistance against bending (flexible types, including macadam) and those which have appreciable "beam" strength (concrete roads, etc.).

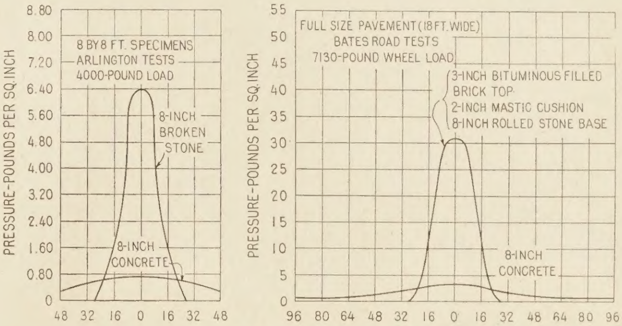

FIGURE 1.—PRESSURE DISTRIBUTION THROUGH VARIOUS TYPES OF ROAD SURFACE

Both field and laboratory observations demonstrate that new water-bound macadams, as compared with rigid types, afford relatively small load distribution. The curves (fig. 1) resulting from tests at the Arlington Experiment Station (*1*)[1] show that a load placed directly on a concrete slab (8 feet square) exerts a unit pressure on a clay subgrade equal to but one-eighth of that exerted by the same load when placed directly on a crushed stone layer of equal area and thickness. The same experiments indicate that a load placed on a concrete slab 4 inches thick exerts a unit pressure upon a clay base equal to that exerted by the same load when placed on a crushed stone layer 42 inches thick.

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[1] References to the bibliography at the end of the article are indicated by an italic numeral inclosed in parentheses.

The curves (fig. 1) resulting from the Bates tests show that a load placed on a full size concrete pavement slab 8 inches thick exerts a unit pressure on the subgrade equal to but one-tenth of that exerted by the same load when placed upon a full-size pavement composed of a 3-inch brick top, a 2-inch mastic cushion, and a macadam base 8 inches thick (*2*, *3*). In the Arlington experiment the load when placed on the macadam surface was distributed over an area 4 feet in diameter, and when placed on the concrete was distributed over an area equal to that of the entire slab (8 by 8 feet). In the field experiment the distance of load distribution was measured only in a longitudinal direction. When placed upon the concrete pavement, the load was distributed through a distance extending more than 8 feet each side of the point of application and, when placed upon the brick pavement, the load was distributed through a distance extending but 2 feet each side the point of application.

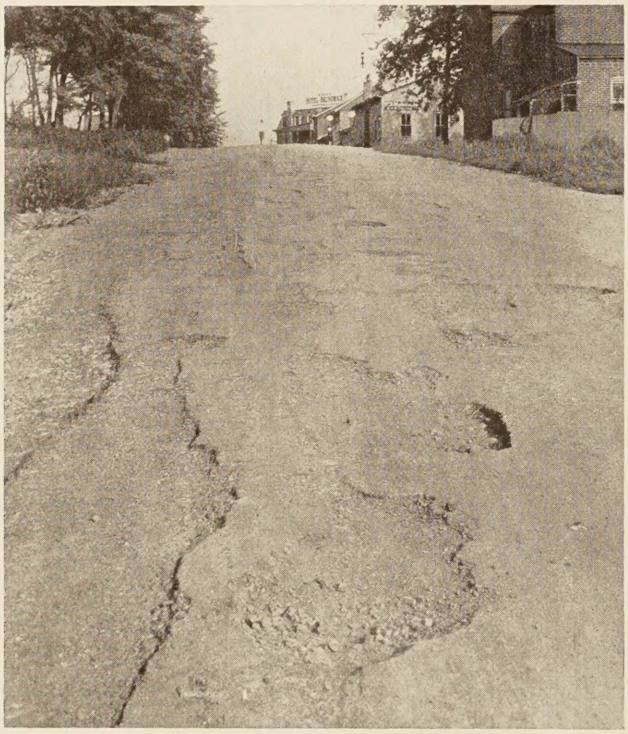


Figure 2.—Surface Failure in a Bituminous Treated Traffic-Bound Road

These tests disclose basic pavement properties as follows: (1) Wheel loads may be spread over much larger areas by concrete slabs than by crushed stone courses or macadams of equal thickness and (2) concrete slabs equal in area to that over which macadams of equal thickness distribute wheel loads transmit a lower maximum unit pressure to the subgrade than the macadams. This is because the total pressure exerted upon the subgrade is distributed more uniformly under concrete slabs than under macadams.

The fact that the pressure transmitted through one pavement was about five times that transmitted through the other is of minor importance, being significant only for the particular conditions of test. The difference between the pressures transmitted through the two pavements may be larger or smaller depending upon the bearing value of the subgrade.

When for instance both pavements are laid on solid rock subgrades they will support loads of equal weight. When in contrast the pavements are raised by frost heave mounds, say about 5 feet apart, so that the intervening portions have no support whatever, the macadam pavement will fail due to its own weight. The concrete pavement, however, will not only support its own weight but in addition will carry appreciable load. Thus the load capacity of both the concrete and the macadam may be approximately equal on subgrades furnishing exceptionally high support. As the subgrade becomes softer and softer, however, the greater becomes the difference between the load capacity of these two pavements.

Thus relative load distribution disclosed by experimental results does not necessarily reflect the relative efficiency of the macadams and the concrete pavements when subjected to service conditions. Furthermore, the crushed stone sections used in the Arlington tests were not true macadam surfaces due to their construction while the macadams in the Bates tests were new and had not been compacted by traffic.

"BEAM" STRENGTH AN IMPORTANT FACTOR

The presence of this property in rigid slabs and its absence in macadam accounts for the difference in load distribution afforded by the two types of pavement. That mixed bituminous pavements also do not possess "beam" strength is indicated by their inability to appreciably retard cracking due to load in rigid bases as shown in the Arlington tests (*4*) and the Bates Road tests.

An analysis of bulletins Nos. 1 to 7 issued by the State of Illinois during the progress of the Bates Road tests furnishes the data contained in Table 1. According to this table, neither the presence of bituminous tops nor the thickness of the bituminous tops exerted a consistent influence upon either the intensity of load required to produce the first corner crack nor the number of corner cracks per unit length of pavement which occurred subsequently in slabs thicker than 4 inches. The extent of breakage, however, was very much reduced by the presence of a bituminous top, was less for the thicker than for the thinner tops, and, other things being equal, it was less in the slabs made from rich mixes than in slabs made from lean mixes.

Table 1.—*Behavior of concrete pavement sections with and without Topeka tops when subjected to truck wheel loads of 2,500 to 8,000 pounds, inclusive.* (*Based on reports on Bates Road tests*)

Section		Concrete		Thickness of bituminous top	Load at which first corner crack occurred	Number of corner cracks per 200 feet of length of pavement	Percentage of pavement requiring replacement
No.	Length	Thickness	Mix				
	Feet	*Inches*		*Inches*	*Pounds*		
61–B	100	4	[1] 1:2:3½	None.	4, 500	10	85. 0
15	200	4	1:2:3½	3	5, 500	3	2. 3
13	200	4	1:3:5	3	5, 500	5	5. 5
14	200	4	1:2:3½	2	5, 500	4	14. 5
12–A	200	4	1:3:5	2	5, 500	6	18. 1
54	200	5	1:2:3½	None.	8, 000	2	49. 0
18	200	5	1:2:3½	3	5, 500	1	0. 9
17	200	5	1:2:3½	2	8, 000	2	1. 0
16	200	5	1:3:5	2	6, 500	3	2. 3
52	200	6	1:2:3½	None.	8, 000	[2] 2	3. 0
20	200	6	1:2:3½	2	8, 000	1	1. 1
19	200	6	1:3:5	2	8, 000	1	1. 3
44	200	7	1:2:3½	None.	6, 500	1	1. 8
21	200	7	1:2:3½	2	6, 500	1	1. 0

[1] With admixtures of hydrated lime, 7 per cent.

[2] Includes one unexplained corner break 6 inches by 6 inches caused by 2,500-pound load.

Additional observations show that the presence of "beam" strength increased the resistance of pavements to ultimate failure. At Arlington (*5*), the load which caused failure in a concrete slab 4 inches thick and 7 feet square produced the same effect in a cement-grout filled brick slab (4 inches thick) when supported by a crushed-stone base 12 inches thick. In the Bates tests, the brick pavement on a macadam base, referred to in Figure 1 previously described, failed under 3,500-pound wheel loads and, contrasted with this, the corresponding concrete pavement section withstood 15,000-pound wheel loads without showing any evidence of distress.

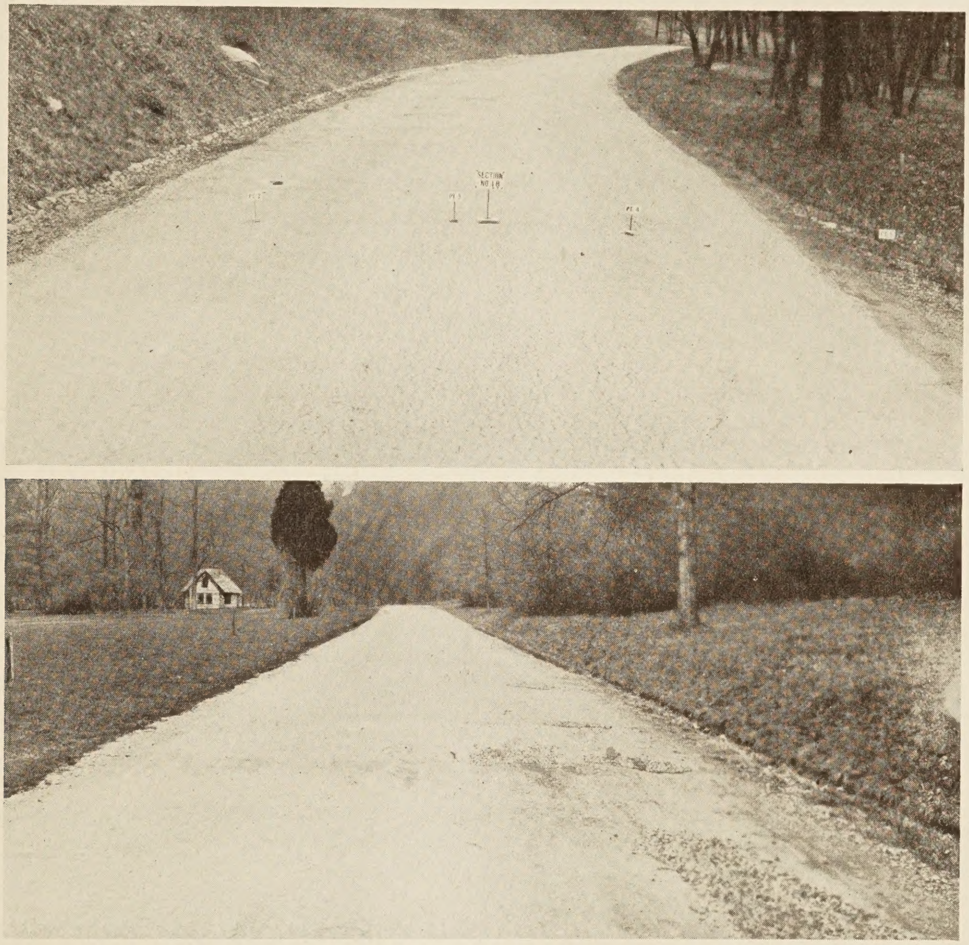


FIGURE 3.—THE UPPER VIEW SHOWS THE BEGINNING OF FAILURE OF MACADAM BASE DUE TO SPONGY SUPPORT, WHILE THE LOWER VIEW SHOWS AN ADVANCED STAGE OF SUCH A FAILURE

PAVEMENT CHARACTERISTICS DISCUSSED

Traffic-bound roads.—Even when laid on soils offering low support traffic-bound roads may automatically develop the high subgrade support which they require because of their lack of "beam" strength. Traffic action continually drives the stone and rock fragments (granular material) into the soil and stabilizes the top layers of the subgrade. Traffic loads are transmitted through the layer of granular material without being widely distributed and gradually increase the density and consequently the supporting value of the subgrade soil and may reduce its ability to take up moisture.

H. J. Kirk, formerly director of the Ohio Department of Highways, describes the behavior of traffic-bound roads in Ohio as follows:

> During the winter, following the application of granular material, traffic pounds the initial layer, about 4 inches thick, down into the subgrade to an extent which indicates a total loss of material. The behavior of an additional 4 inches of granular material applied subsequently shows that instead of being wasted, the initial layer served to increase the subgrade support to an extent which furnished stability in the resulting road surface greatly exceeding that furnished by the same amount of granular material when applied or compacted in any other way.

Thus traffic-bound roads may consist of a combination of subgrade treatment and road surfacing. This accounts for their high load capacity (*6*).

Appreciable vertical deflections, however, caused by local frost heave, increased wheel loads, or the development of soft spots in the subgrade are productive of failure in traffic-bound roads. Surface abrasion (fig. 2) when not prevented by proper maintenance measures, may continue downward and cause the total destruction of these roads regardless of the character of subgrade support.

Water-bound macadam pavement.—Increase in subgrade support due to the compaction of the underlying soils occurs also under macadams. The stability of these pavements depends upon the extent of initial compaction, the traffic action which occurs prior to the application of a surface treatment (*6*), their age, and upon the materials of which they are composed.

Macadams may adjust themselves to appreciable subgrade deformations when caused by the slow and gradual compaction of the underlying soil. Abrupt

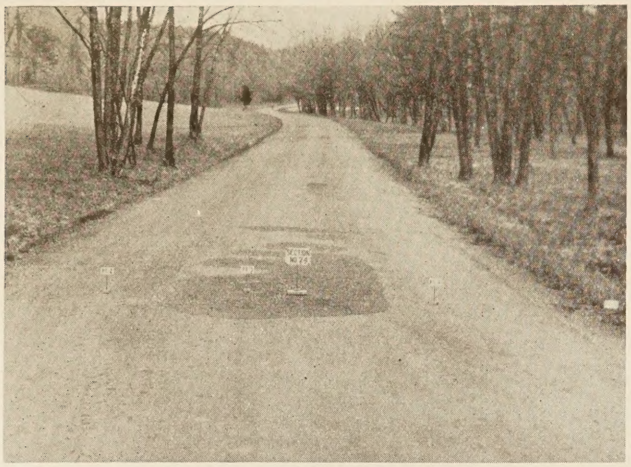


Figure 4.—Failure Localized to Soft Spot in Subgrade

deformations similar to those which cause failures in traffic-bound roads may cause failure also in macadams. Difficulty may be expected also when spongy subgrade support prevents the pavement from receiving the required amount of compaction during construction; when clay works up into the interstices of pavements containing large voids; and when disintegration beginning with surface abrasion is not prevented. Figure 3 illustrates failures of water-bound macadam base under conditions which indicate spongy subgrade support. A ruptured bond, extending generally throughout the width of the pavement, is characteristic of this type of failure. In contrast, the failures shown in Figure 4 indicate localized soft spots in the subgrade, as most of the pavement is intact.

Bituminous pavements.—The binder in bituminous pavements consists entirely of bituminous materials. These pavements are tough, wear resistant, plastic, and, in the thicknesses generally used, require strong base support. They withstand considerable and repeated vertical movement without failure and therefore afford considerable protection to broken concrete bases.

Failure in bituminous pavements due to subgrade influence reflects only the behavior of their bases. Figure 5 shows the results of a macadam base failure and a concrete base failure under bituminous pavements.

Rigid pavements.—Concrete pavements are highly resistant to surface abrasion and possess "beam" strength. They withstand appreciable heave if it is uniform, but cracking occurs where there is abrupt variation in subgrade support. Internal stresses, due to variation in temperature and moisture content, also cause cracks. Regardless of the cause, cracking in concrete slabs can be of four kinds—longitudinal, transverse, corner, and irregular.

Longitudinal cracking may occur as a single crack which is generally near the center of a full-width road slab or such a crack accompanied by additional ones near the center of the slabs formed by the crack near the center. The former type will hereafter be referred to as longitudinal cracks and the latter type will be referred to as secondary longitudinal cracks.

The occurrence of longitudinal cracks depends to a large extent upon the character of subgrade support. They seldom occur in pavements laid on sands and gravels (*7*) or on highly stable subgrades of several of the Southern States, notably North Carolina. Other things being equal, they occur to less extent in pavements laid on porous sub-bases than in those laid on the natural soil (*8*).

Data obtained in California (*9*) during the dry season of the year and illustrated in Figure 8 show why longitudinal cracks occur primarily in pavements laid on unstable soils. The lines representing equal moisture content in Figure 6 were probably parallel to the ground surface prior to constructing the pavement. These lines in the sand-clay subgrade have not varied in this respect, thus indicating that expansion or shrinkage in the subgrade due to variation in moisture content is of uniform intensity throughout the width of the pavement. Such being the case, the pavement would not be expected to crack longitudinally. The lines of equal moisture content in the adobe and clay subgrade, however, have not remained parallel to the ground (or road) surface, thus indicating that expansion and shrinkage in the subgrade varies considerably in extent throughout the width of the pavement.

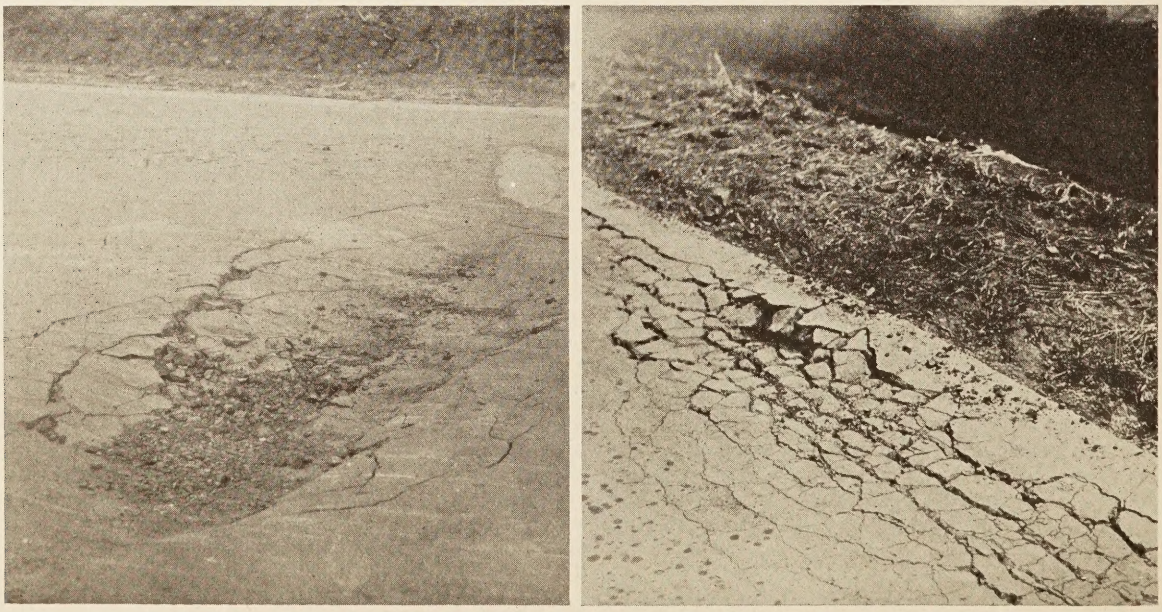
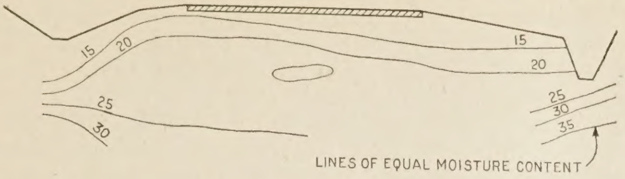
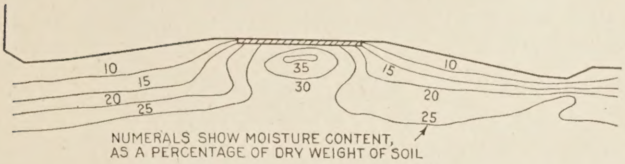


Figure 5.—Result of Failure of Concrete Base (Left) and Macadam Base (Right). In Both Cases the Surface is of the Flexible Type



SAND CLAY SUBGRADE


10
15
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20
25
NUMERALS SHOW MOISTURE CONTENT,
AS A PERCENTAGE OF DRY WEIGHT OF SOIL

ADOBE AND CLAY SUBGRADE

FIGURE 6.—LINES OF EQUAL MOISTURE CONTENT IN SUBGRADES OF DIFFERENT CHARACTER

The difference between the moisture content of the adobe and clay subgrade under the center and under the edges of the pavement is, according to Figure 6, about 15 per cent of the weight or about 40 per cent of the volume of the dry soil. When subjected to conditions causing maximum shrinkage the soil under the edges of the pavement will have a volume equal to but 70 per cent of that of the soil under the center of the pavement. In some instances in California shrinkage of this type permits open spaces as much as 5 inches in depth to occur between the pavement edge and the subgrade. This condition of high support existing under its center and the little or no support existing under its edges will cause the pavement to crack longitudinally. Cracking due to this cause is shown in Figure 7, although in these particular cases the longitudinal cracks do not happen to be near the center of the pavement.

In order to facilitate subsequent analyses transverse cracks will be referred to as primary transverse cracks, secondary transverse cracks, and tertiary transverse cracks. Slabs formed by the occurrence of primary transverse cracks will be referred to as primary road slabs and those formed by the occurrence of secondary transverse cracks will be referred to as secondary road slabs. Subdivision of secondary slabs by tertiary transverse and secondary longitudinal cracks will be referred to as "breakage."

Primary transverse cracks are those which occur at an average interval of not less than 18 feet in full width concrete road slabs of appreciable age (*7*). Their occurrence may be greatly influenced by the quality of the concrete, by the mechanical friction which occurs between the bottom of the slab and the subgrade (*10*), and by the uniformity of subgrade support. They may occur less frequently in pavements laid on soils affording low but uniform support than in those laid on subgrades with high but less uniform support. This is demonstrated in an unpublished theoretical analysis by Dr. H. M. Westergaard.[2]

According to this analysis the stresses in the concrete slab due either to warping of the slab or to nonuniform volume change of the subgrade increase with the stiffness of the subgrade. Such cracks, due to lack of uniformity in support (fig. 8), may occur more or less irregularly in pavements, and those caused by frictional resistance may occur approximately parallel to each other and perpendicular to the center line (*10*).

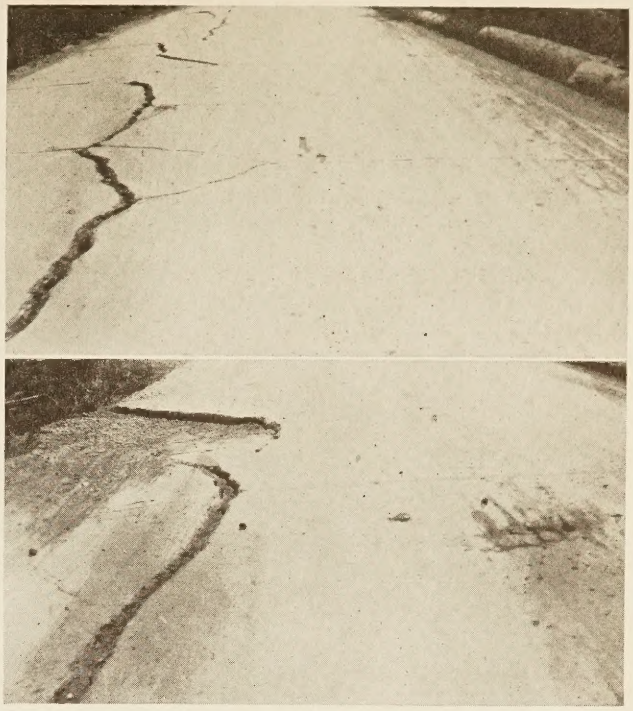

FIGURE 7.—CRACKING ON A FILL MADE WITH CLAY HAVING HIGH SHRINKAGE PROPERTIES

Secondary transverse cracks (single cracks in primary slabs) seem to be caused by some combination of load and support. Their occurrence, when perpendicular to the center line of the pavement, may indicate low subgrade support, and when irregular may indicate a lack of uniformity in subgrade support. The lower picture of Figure 8 shows settlement in a fill which caused secondary transverse pavement cracking.

"Breakage" due to load occurs on account of low subgrade support. This was clearly brought out in surveys by the highway research board (*7*) and experiments performed at Arlington. According to the Arlington test data (*4*) summarized in Table 2, the ultimate resistance of slabs (7 feet square) to the occurrence of breaking differs considerably depending upon whether they are laid on a drained or an undrained subgrade.

TABLE 2.—*Average load capacities of nonreinforced concrete slabs 7 feet square, when laid on clay subgrades drained and undrained. (Impacts applied at corner and edges of slabs, Arlington tests)*

Slab thickness. inches	Mix	Breaking loads [1]	
		Wet subgrade	Drained subgrade
		Pounds	*Pounds*
4	1:1½:3	(2)	13, 650
6	1:1½:3	12, 825	[3] 20, 000
[4] 6	1:1½:3	10, 675	22, 225
8	1:1½:3	25, 900	42, 040
[4] 8	1:1½:3	25, 475	38, 800
[4] 6	1:3:6	(5)	18, 430
6	1:3:6	9, 580	[3] 18, 300

[1] Unless otherwise noted each value is the average of two tests, one with the load applied at the edge and the other with the load applied at the corner of the slab.
[2] Broke under static load varying between 2,000 and 8,000 pounds.
[3] One edge test.
[4] Covered with bituminous tops.
[5] Six out of eight slabs tested broke under static loads varying between 2,000 and 8,000 pounds. The other two tests, both for edge loading, averaged 10,105 pounds.

Increasing the subgrade soil support by admixture of Portland cement (1:14 and 1:28 about 12 inches deep) increased the load capacity of superimposed slabs, when subjected to impacts, by 50 to 65 per cent (*11*).

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[2] This analysis will appear in the June issue of Public Roads.

Breakage of considerable extent occurs primarily in pavements less than 6 inches thick (*7*) (see Table 1).

Corner cracks may be due to variation in the width of expansion joints or to the presence in them of foreign materials such as stones and mortar, to the improper use of longitudinal steel in bond (*12*) and to traffic. Corners broken off by expansion of the concrete are generally small in area, and have sides ranging from 6 to 18 inches in length. Broken corners due to continuous longitudinal steel in bond in the absence of expansion joints, occur either along the edges or the center joint, are usually held tightly in place, and may have a longer leg transversely than longitudinally. These are illustrated in Figure 9. Corner cracks caused by longitudinal steel in bond extending across transverse expansion joints may constitute failure requiring replacement. Corners broken off by traffic generally have equal legs 2 to 4 feet long and in many cases are depressed. The upper picture of Figure 9 illustrates a corner crack caused by traffic. The term "corner cracks" when used subsequently refers only to those caused by load.

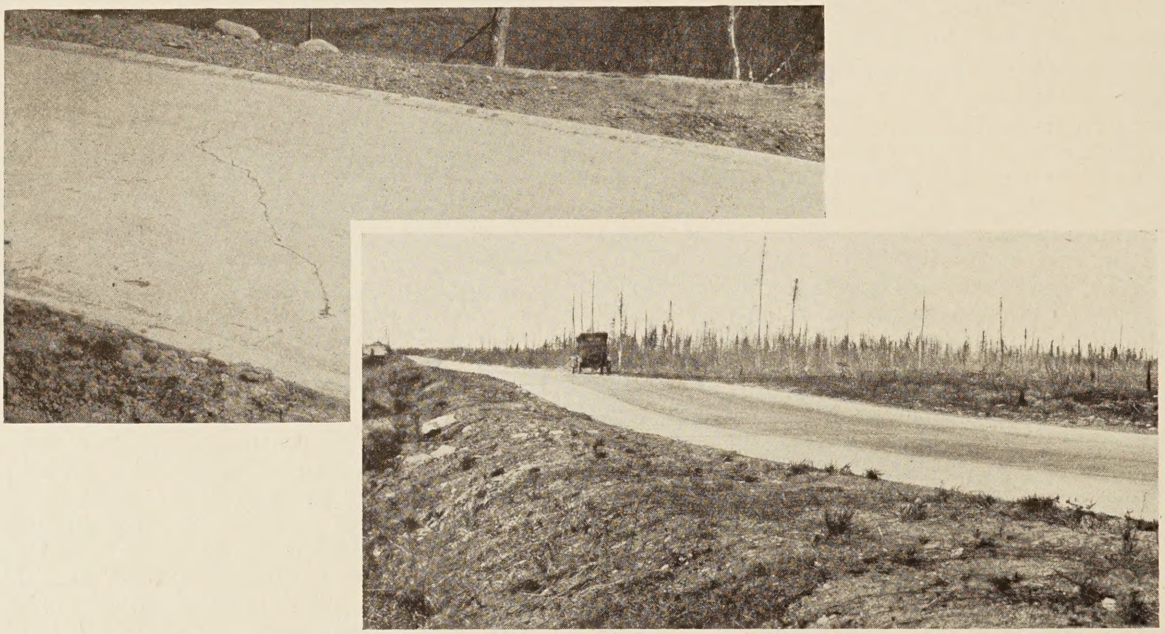


Figure 8.—The Upper Picture Shows Cracks in Concrete Road Caused by Lack of Uniformity in Subgrade Support. The Lower Picture Shows Transverse Cracks at Short Intervals Due to Subgrade Settlement

Excepting when the corners of full width concrete road slabs are warped upward (at night and after showers in the day time) the occurrence of "corner cracks," like "breakage," seems to depend upon the intensity of subgrade support.

Irregular cracks may consist of short single lines which occur intermittently in road slabs or which, when more or less connected, may extend for appreciable lengths in road slabs.

They may be caused by shrinkage in the concrete due to mixing, curing or material variables, or by nonuniform expansion in the subgrade soil. In any case they occur at least in microscopic dimensions before the concrete has set. Both W. C. McNown, of the University of Kansas, and Mr. F. V. Reagel, of the Missouri State highway department, observed cracks of this type occurring in parts of the pavement located over ruts in the subgrade which had been filled with dry soil immediately prior to constructing the pavement. (See fig. 10.)

The faulting of cracked slabs is dependent upon subgrade support. The separation of cracked slabs (fig. 11) or the widening of center joints, when occurring generally, may be due to movement of the subgrade.

The intimate relations existing between pavement condition and subgrade support causes one to inquire next into the variables upon which the supporting properties of the subgrade depend.

FIELD INVESTIGATIONS SHOW THAT SUBGRADE VARIABLES INFLUENCE PAVEMENT CONDITION

According to the older conceptions derived from experience, the quality of the subgrade support should essentially depend on the clay content. Thus the results furnished by the California survey (*13*), and by investigations of the highway research board (*7*), show that the occurrence of "breakage" in relatively thin concrete pavements is influenced by the clay content of the subgrade soil.

A. C. Rose, in the Pacific Northwest (*14*), and C. L. McKesson in California (*15*), also found that the clay content of the subgrade soil influenced pavement condition to marked extent.

Additional data show that, other things being equal and especially in the absence of frost action, the presence of particles smaller than 0.005 millimeter in diameter contributes to the occurrence of pavement failures caused by low intensity of subgrade support. The results furnished by the subgrade surveys now in progress, however, disclose the identity of factors other than the clay content of the subgrade soil which may exert an influence of primary importance upon the occurrence of failure in pavements.

For instance, failures due to low intensity of support, according to data now available, may be caused merely by the presence of highly porous materials such as diatoms or by a high moisture content in the subgrade soil. Failures due to spongy support may be caused

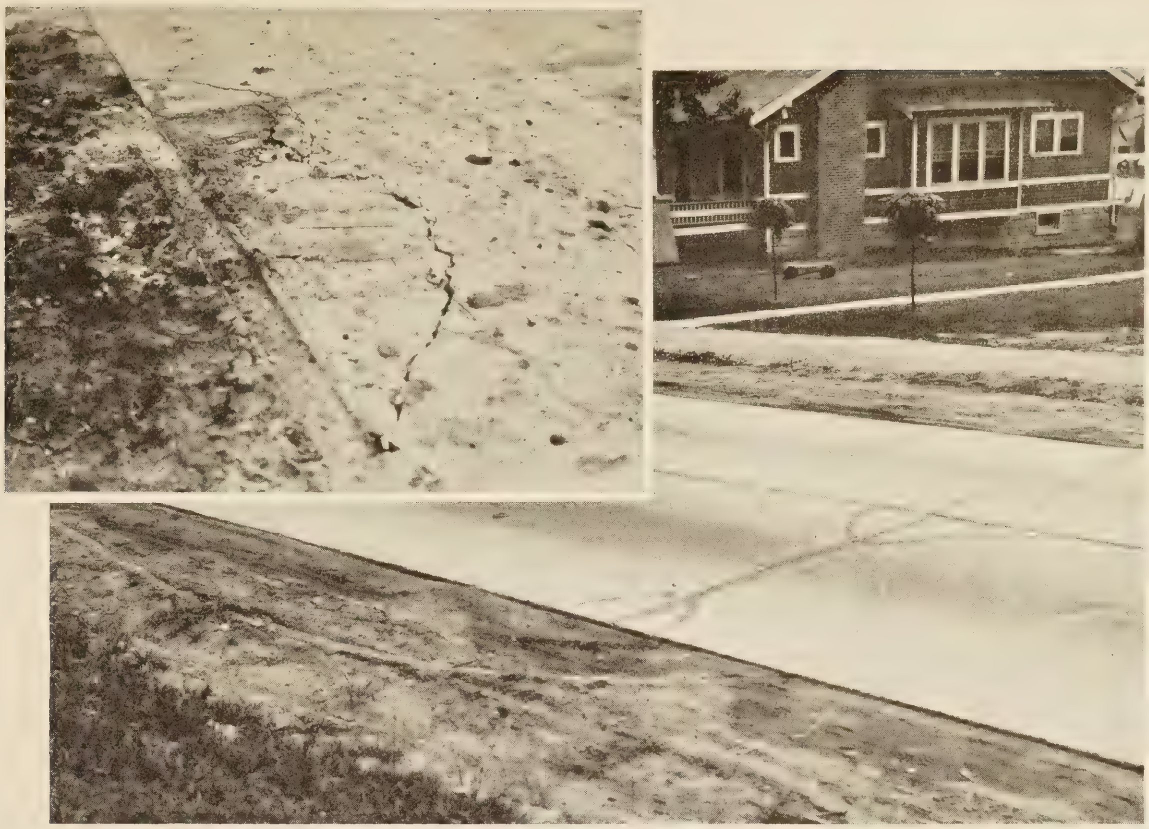


FIGURE 9.—CORNER CRACKS ALONG THE EDGES AND CENTER JOINT CAUSED BY HEAVY LONGITUDINAL REINFORCEMENT IN BOND. THE UPPER PICTURE SHOWS A CORNER BREAK CAUSED BY TRAFFIC AND FOLLOWED BY PROGRESSIVE BREAKING DOWN

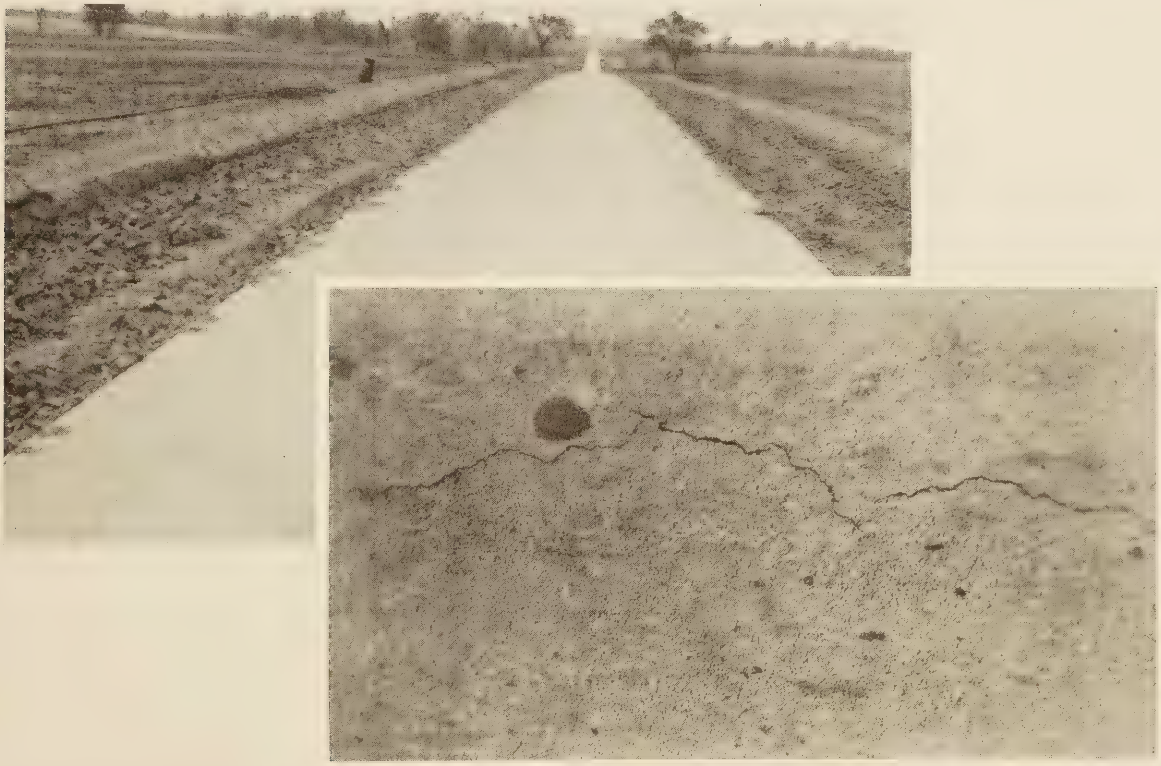


FIGURE 10.—UPPER: CRACKING LOCALIZED IN PORTIONS OF CONCRETE LAID OVER WHEEL PATHS IN SUBGRADE. LOWER: IRREGULAR CRACKING CAUSED BY EXPANSION IN THE SUBGRADE COMBINED POSSIBLY WITH SHRINKAGE IN THE CONCRETE DUE TO LOSS OF WATER FROM THE CONCRETE BEFORE SETTING

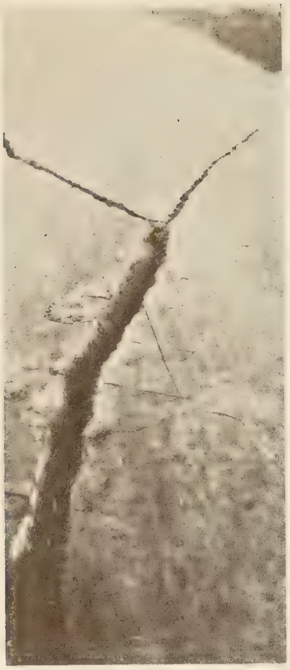


Figure 11.—Separation of Slab Fragments Due to Subgrade Conditions

solely by the presence of mica in the subgrade soil. Failures due to lack of uniformity in support may be caused by variation in the soil directly under the pavement or at some distance beneath the pavement.

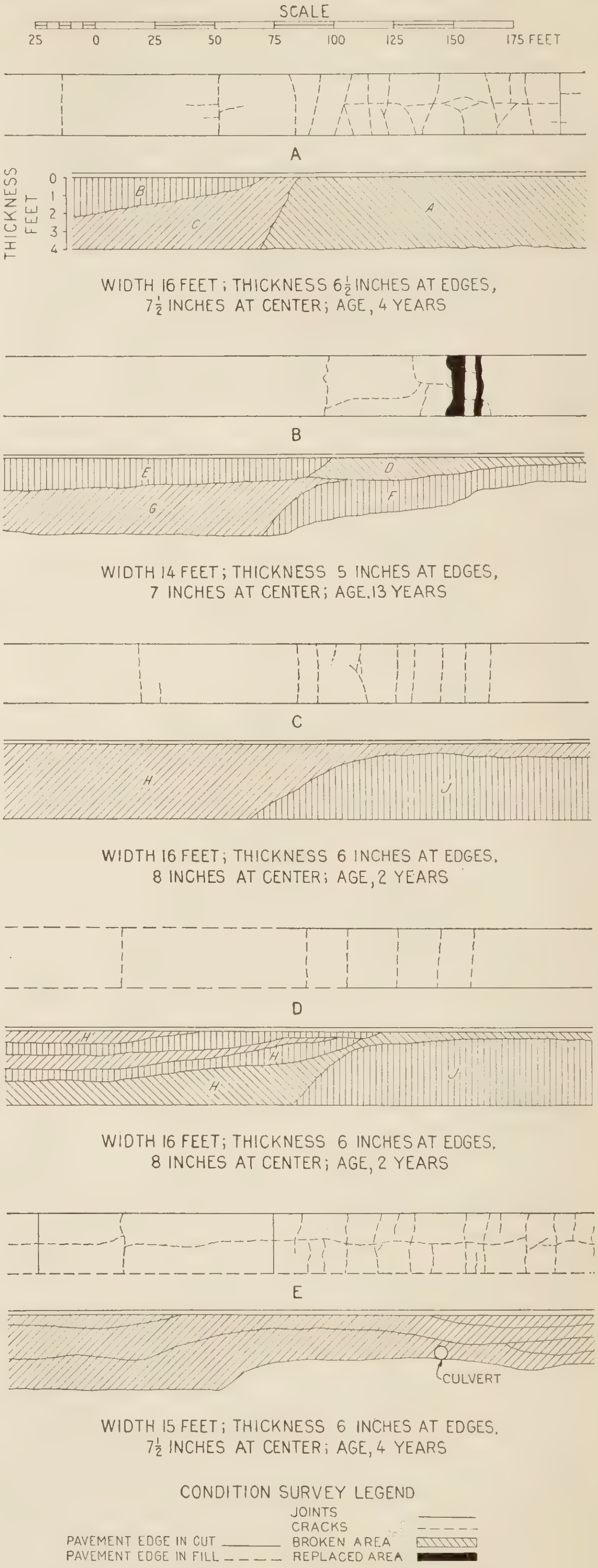
In Figure 12–A,[3] the badly cracked portion of the pavement rests upon a silty soil *A*, which contains some clay and an appreciable percentage of diatoms. This soil has high shrinkage and plasticity; when dry it is highly porous and has a strong affinity for water but when wet it is very unstable. Soil type *B*, upon which part of the good portion of the pavement rests, is a fine, sandy loam and has medium shrinkage and plasticity. Soil *C* lacks the diatom content but otherwise is similar to *A*. Thus, in this case, the presence of diatoms in the subgrade soil seems to have influenced the extent of cracking in the pavement.

The failure shown in Figure 3 and referred to previously, was attributed primarily to the presence of large mica flakes in a very fine sand and silty soil.

In Figure 12–B the cracked portion of the pavement rests directly on a soil, *D*, which varies from a fine sandy loam to a clay and which has high shrinkage and medium plasticity. The good portion of the pavement rests directly upon a soil mixture, *E*, which has negligible shrinkage and plasticity. The underlying soils, *F* and *G*, are silty clays which have medium shrinkage and plasticity. In this case, the clay content of the subgrade soil seems to have influenced the extent of cracking in the pavement.

In Figure 12–C, both the cracked and good portions of the pavement rests directly upon a soil, *H*, which has medium shrinkage, medium plasticity and which, due to its structure, is permeable. Under the cracked portion, however, the underlying layer, *J*, is very compact and impervious (due to its laminated structure), has medium shrinkage and plasticity, and lies very close to the pavement. Because of the approximately curved profile of the underlying layer, *J*, the thickness of the upper layer, *H*, is variable. Under these conditions, the porous upper layer, due to variation in moisture content accompanied by expansion and shrinkage, may furnish the cracked portion of the pavement with support which lacks uniformity. Also, at this location, the impervious underlying layer, *J*, prevents the escape of water downward and thus may cause loss of stability in the upper layer, *H*, due to supersaturation.

A similar subgrade condition accompanied by similar cracking in the pavement, is shown in Figure 12–D.



WIDTH 16 FEET; THICKNESS $6\frac{1}{2}$ INCHES AT EDGES, $7\frac{1}{2}$ INCHES AT CENTER; AGE, 4 YEARS

WIDTH 14 FEET; THICKNESS 5 INCHES AT EDGES, 7 INCHES AT CENTER; AGE, 13 YEARS

WIDTH 16 FEET; THICKNESS 6 INCHES AT EDGES, 8 INCHES AT CENTER; AGE, 2 YEARS

WIDTH 16 FEET; THICKNESS 6 INCHES AT EDGES, 8 INCHES AT CENTER; AGE, 2 YEARS

WIDTH 15 FEET; THICKNESS 6 INCHES AT EDGES, $7\frac{1}{2}$ INCHES AT CENTER; AGE, 4 YEARS

Figure 12.—Soil Profiles Compared with Pavement Condition

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[3] The soil profiles in Figure 12 were furnished by W. I. Watkins, of the U. S. Bureau of Chemistry and Soils, which is cooperating with the Bureau of Public Roads in the latter's subgrade surveys.

In this case, however, the good portion of the pavement rests upon a fill while the pavement shown in Figure 12–C is all in cut. The cracking shown in Figures 12–C and 12–D seems to be due more to field arrangement and structure than to the raw materials of the subgrade soils.

In Figure 12–E, all of the soils are fine sands or fine sandy loams and have negligible shrinkage and plasticity. Cracking here seems to be due to settlement in the side-hill fill, caused possibly by the failure of the culvert located under the center of the cracked portion of the pavement to furnish adequate drainage facilities.

Side-hill fills constructed of material which is less permeable than the soil which supports the fill material and part of the pavement, subgrades in which the soil changes appreciably in condition or character, and soils composed of widely different materials as, for instance, clays interspersed with sand pockets, and sands containing thin, laminated clay layers, afford conditions of support especially productive of cracking in pavements.

In this connection attention may be called to the fact that clay fills are often very much less permeable to a depth of several feet below the surface than the same clay in its natural condition. This is due to the closing of root holes and other openings which exist in the undisturbed deposit.

Thus, according to the preceding discussions, pavement behavior may depend upon the character of the subgrade soil material (raw constituents), upon the structure of the soil in its natural state (dense or loose, homogeneous or full of cracks or root holes), upon the soil profile (variation in depth of the different soil zones and the relative occurrence of permeable and impermeable strata) upon adjacent topography (through its influence upon the occurrence of surface and underground water), upon climatic conditions (well distributed or intermittent occurrence of rainfall and presence or absence or frost action), or upon any combination of these variables.

The number and variety of these influencing factors clearly indicate that they must be investigated according to some coherent and logical procedure, otherwise the ensuing information, because of its complexity, will be of limited usefulness in practice.

Therefore, in order to facilitate a study of these different variables with respect to the relative importance of the influence which they exert upon the supporting properties of subgrades and in order to facilitate the application of the ensuing information in practical road construction, it becomes desirable to arrange the subgrades in groups according to outstanding performance with regard to pavement behavior.

SUBGRADES GROUPED ACCORDING TO PERFORMANCE

In determining a satisfactory basis for grouping subgrades, their properties as disclosed by both laboratory tests and field surveys must be considered.

Laboratory test results disclose the degree to which certain physical properties are manifested when soils are subjected to arbitrary laboratory conditions, and therefore furnish a means merely for identifying the raw constituents of soils and determining the influence which these raw constituents are apt to exert upon such physical characteristics as shrinkage, expansion, plasticity, elasticity, water capacity, permeability, etc., and the extent to which admixtures or treatments may be effective for changing these characteristics. The more the field behavior depends upon the raw constituents of soils, the more self-sufficient are laboratory tests for explaining the behavior of subgrades and, because of this, laboratory tests explain with greater accuracy the performance of subgrades consisting of coarse-grained soils than of those consisting of fine-grained soils.

Thus, for instance, if a soil, according to test, shows neither plasticity nor shrinkage, and if in addition its mechanical analysis indicates a coarse-grained, well-graded material with some silt in it, it will represent a firm subgrade in the field.

On the other hand, the more the fine-grained constituents (clay content) of a soil dominate, the greater the possible variation in behavior of exactly the same soil under field conditions and the more do the data obtained in the laboratory require supplementing by field observation.

Therefore, the tentative grouping of subgrades takes into consideration both their raw constituents and their performance under field conditions. The extent to which laboratory tests serve for identifying subgrades and the extent to which subgrades with equal laboratory identification may vary in performance under field conditions is indicated by the following discussion of the individual groups.

It should be remembered that no attempt is made to present a rigid, permanent classification. The group numbers are used merely for convenience and have no other significance. When the character of the subgrade changes, there is a weak spot although the subgrades on both sides of the boundary may be good. Therefore, when attempting to arrange the subgrades in groups, uniform support (A) is distinguished from nonuniform support (B).

Within Group A the subgrades are arranged according to those characteristics of the soils which are most conspicuous in the performance of subgrades.

UNIFORM SUBGRADES (GROUP A) ARRANGED ACCORDING TO CHARACTERISTICS CONSPICUOUS IN PERFORMANCE

Group A–1.—Well graded material, coarse and fine, excellent binder. Highly stable under wheel loads, irrespective of moisture conditions. Functions satisfactorily when surface-treated or when used as a base for relatively thin wearing courses. Represented by the excellent topsoils of Georgia.

Group A–2.—Coarse and fine materials, inferior binder. Highly stable when fairly dry. Apt to soften at high water content caused either by rains or by capillary rise from saturated lower strata when an impervious cover prevents evaporation from the top layer.

Group A–*3.*—Coarse material only, no binder. Lacks stability under wheel loads but unaffected by moisture conditions. Furnishes excellent support for flexible pavements of moderate thickness and for relatively thin rigid pavements. Represented by the Florida sands.

Group A–4.—Silt soils without coarse material, and with no appreciable amount of clay. Apt to absorb water very readily in quantities sufficient to cause rapid loss of stability even when not manipulated. When dry or damp, presents a firm riding surface which rebounds but very little upon the removal of load. Apt to cause cracking in rigid pavements due to frost heaving and failure in flexible pavements due to low support. Represented by the New Hampshire silts.

Group A–5.—Similar to Group A–4, but furnishes highly elastic riding surfaces with appreciable rebound upon removal of load even when dry. Elastic properties interfere with proper compaction of macadams during construction and with retention of good bond afterwards. Represented by highly micaceous soils of North Carolina.

Group A–6.—Clay soils without coarse material. In stiff or soft plastic state absorb additional water only if manipulated. May then change to liquid state and work up into the interstices of macadams. Furnish firm support essential in properly compacting macadams only at stiff consistency. Deformations occur slowly and removal of load causes very little rebound. Shrinkage properties combined with alternate wetting and drying under field conditions are apt to cause cracking in rigid pavements. Represented by the Mississippi gumbo.

Group A–7.—Similar to Group A–6, but when moist, deforms quickly under load and rebounds appreciably upon removal of load. Thus, lacks firmness in support, similar to subgrades of Group A–5. Alternate wetting and drying under field conditions leads to even more detrimental volume changes than in Group A–6 subgrades. May cause concrete pavements to crack before setting and to crack and fault afterwards. Represented by Illinois gumbo and typical adobes.

Group A–8.—Very soft peat and muck incapable of supporting a road surface without being previously compacted or displaced by a fill. Represented by Michigan peat bogs.

NONUNIFORM SUBGRADES GROUPED

Soils of this group cause concrete pavements to crack or fault excessively and flexible types to fail or to develop rough riding surfaces.

Group B–1.—Nonuniform natural ground due to abrupt variation in soil characteristics, or soil profile, or to frequent change in field conditions.

Group B–2.—Nonuniform subgrade due to nonuniform composition of fill.

Group B–3.—Nonuniform subgrade consisting in part of natural ground and part of fill materials.

GROUP A-1 SUBGRADES CHARACTERIZED BY HIGH STABILITY

The high stability of the Group A–1 subgrades is due to the presence of coarse material in an amount sufficient to furnish high supporting value and fine particles in quantity and quality sufficient to furnish a fairly water-resistant binder which prevents lateral displacement of the soil when subjected to load.

The mechanical composition of an ideal soil of class 1, according to Dr. C. M. Strahan (*16*) is as follows: "Clay 12 to 18 per cent; silt, 5 to 15 per cent; total sand, 65 to 80 per cent, and sand above No. 60 sieve, 45 to 60 per cent. When coarse material (that retained on No. 10 sieve) is present or is added to a good soil mortar in appreciable amount, 10 per cent or more, the hardness and durability of the slab is increased, and continues to increase until the full type of gravel slab is reached. Coarse material is most effective when present in graded sizes from one inch downwards. Micaceous, feldspathic or slaty types are objectionable because of their softness." In Dr. Strahan's report "clay" refers to particles less than about 0.02 millimeters in diameters; silt, to particles whose diameters vary between 0.02 and 0.07; and sand to particles larger than 0.07 millimeters in diameter. Subsequently in this report "clay" refers to particles less than 0.005 millimeters in diameter; silt to particles whose diameters vary between 0.005 and 0.05; and sand to particles larger than 0.05 millimeters in diameter.

Translating his statement into the terms used by the Bureau of Public Roads (clay 0.005 millimeters, silt between 0.005 and 0.05 millimeter, and sand for particles larger than 0.05 millimeter), we arrive at the following approximate composition of the material: Clay less than 5 per cent, silt 9 to 32 per cent, sand total 63 to 86 per cent, and sand above No. 60 sieve 45 to 60 per cent. The uniformity coefficient of such a material should be greater than 15 and its effective size in the vicinity of 0.01 millimeter.[4]

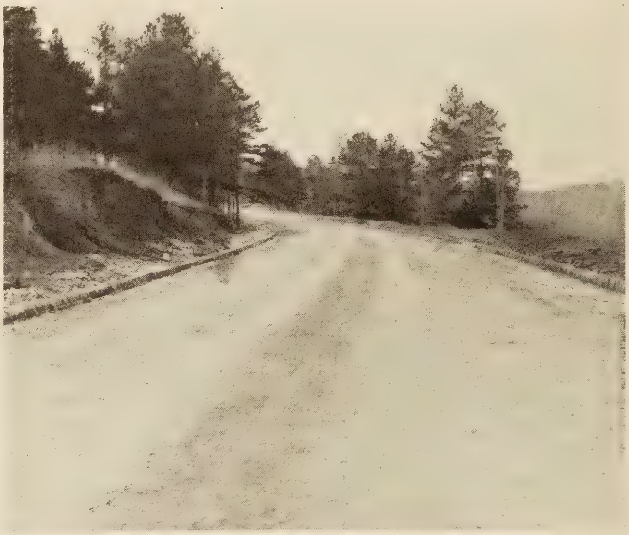

Figure 13.—Group A–1 Soil, a Stable Mixture of Sand and Clay

According to C. L. McKesson (*17*) good binders usually have low moisture carrying capacities and low lineal shrinkage values (*14*) and according to Dr. C. M. Strahan they have also high adhesive properties (*18*). The latter suggests kaolin, which according to laboratory tests results has low shrinkage and medium plastic properties, as an ideal binder.

The high stability which identifies the Group A–1 subgrades depends upon characteristics of the raw materials and is disclosed in the laboratory by the mechanical analysis which informs us about the quantity of binder, and the plasticity and shrinkage tests which inform us as to the quality of the binder.

In nature these subgrades are represented by many bank-run gravels, natural sand-clays and certain topsoils such as are found in parts of North Carolina and Georgia. Their stability when subjected to traffic is illustrated in Figure 13.

Their performance is influenced by only two field conditions: Position of the ground-water level and frost action. If the ground-water level rises close to the ground surface, frost heave is apt to occur in the more uniform varieties of these subgrades. More or less horizontal layers of ice are formed, while the layers of soil located between the ice layers seem to come under compression. When the ice thaws, the soil is apt to disintegrate temporarily. Group A–1 subgrades make excellent fills in which settlements are insignificant.

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[4] Effective size: The size of grain than which there are 10 per cent, by weight of the particles smaller and 90 per cent larger is considered to be the effective size.

Uniformity coefficient: The ratio of the size of grain than which there are 60 per cent of the sample finer to the size than which there are 10 per cent finer. (Allen Hazen Report of the Massachusetts State Board of Health, 1892, p. 541.)

GROUP A-2 SUBGRADES MORE AFFECTED BY MOISTURE THAN THOSE OF GROUP A

Subgrades of this group may contain either more binder or an equal amount of binder of poorer quality than those of Group A–1. Although a Group A–2 subgrade, during the dry season, could hardly be distinguished from a Group A–1 subgrade, the former, by its softening, can easily be distinguished from the latter during rainy weather or when covered with an impervious top.

Since the softening of the binder is quite obviously a characteristic of the raw material, the Group A–2 subgrades may be readily identified by the same laboratory tests which identify the Group A–1 subgrades.

The field conditions which exert an influence on the performance of these subgrades are position of the ground-water level, frost action and intensity, and distribution of rainfall. The position of the ground-water level and frost action exert an influence similar to that noted for the Group A–1 subgrades. In addition, proximity of the ground-water level to the ground surface increases the tendency of these subgrades to soften when covered with impervious tops. Also, the more wet weather there is the less serviceable are these subgrades.

Subgrades varying with respect to size of particles and grading from those of Groups A–1 and A–2 approach those of other groups as follows: With decreasing uniformity and decreasing quantity of binder, Group A–3; with decreasing uniformity and increasing silt quantity, Group A–4; and with increasing clay content, Group A–6.

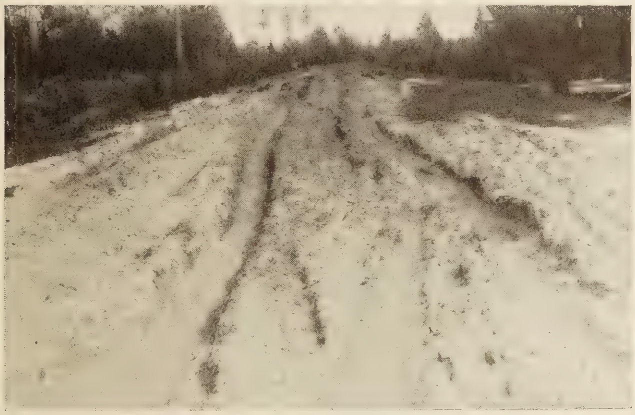

Figure 14.—Group A–3 Soil, Cohesionless Sand

GROUP A-3 SOILS LACKING IN COHESION

These subgrades differ from those of Groups A–1 and A–2 inasmuch as they lack the fine fraction (binder, particles less than 0.05 millimeter in diameter) required to prevent lateral displacement of the coarse particles under load. Hence when traveling over the surface of such a subgrade, the wheels sink into the loose material. At the same time, the supporting qualities of the same material when loaded over a very large area are very good.

All of the Group A–3 subgrades have the following properties: Effective size not less than 0.1 millimeters; no shrinkage; and no plasticity. The higher the uniformity coefficient and the sharper the grains, the more stable is the material.

These properties all depend upon characteristics of the raw materials and can be identified in the laboratory by the mechanical analysis, and the plasticity and shrinkage tests.

Cohesionless sands and gravels represent these subgrades in nature. Their behavior under traffic is illustrated in Figure 14.

Field conditions are apt to influence the quality of these subgrades in the following manner: The same sand may occur in nature either in a dense or in a loose state. The denser the texture the greater is the supporting power of the material. If the ground-water level rises close to the surface, the supporting power of the subgrade decreases very considerably.

Due to their great permeability, the Group A–3 subgrades are subject to frost heave only in very rare instances. They make very stable fills irrespective of moisture conditions.

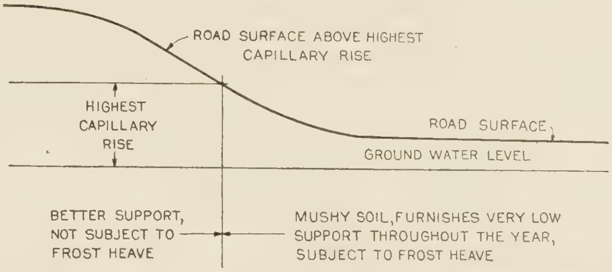
ROAD SURFACE ABOVE HIGHEST CAPILLARY RISE
HIGHEST CAPILLARY RISE
ROAD SURFACE
GROUND WATER LEVEL
BETTER SUPPORT, NOT SUBJECT TO FROST HEAVE
MUSHY SOIL, FURNISHES VERY LOW SUPPORT THROUGHOUT THE YEAR, SUBJECT TO FROST HEAVE

Figure 15.—Longitudinal Profile Showing Effect of Elevation of Ground Water Level on the Quality of Group A–4 Subgrades

GROUP A-4 SOILS LACKING IN BOTH FRICTION AND COHESION

These subgrades differ from those of Group A–1 inasmuch as they lack coarse particles in amounts sufficient to make them stable when wet, and, inasmuch as they lack very fine particles (clay) in sufficient amount to produce appreciable cohesion.

All of the Group A–4 subgrades have the following laboratory properties in common: Uniformity coefficient less than 15; effective size from about 0.01 to 0.10 millimeters; plasticity index seldom above 10 and generally 0. Shrinkage limit below 30. Soils of this group can therefore be identified by the following tests: Wet mechanical analysis (Wiegner or Boyoucos method) and the Atterberg plasticity tests.

They are represented in nature by the very fine sands and silts which occur generally and have been noted particularly in New Hampshire, Maryland, Ohio, and in the frost boil areas of Iowa and Minnesota.

Field conditions may affect the quality of these subgrades in different ways. The same soil may be dense or loose, which in turn affects the supporting quality both in a dry and in a wet state. The closer the particle are packed together, the greater is the supporting power of the same soil (settlement under the same load acting on the same area). If the ground-water level is close to the surface, the average supporting power of the soil throughout the year will be very much lower than if the ground-water level occupies a low position. Furthermore, at high ground-water level, important frost heave must be expected, while it may be absent if the ground-water level is low. This effect of the field conditions is illustrated by Figure 15.

Furthermore, the somewhat plastic (very fine grained) varieties of the Group A–4 subgrades can be either feebly permeable or fairly permeable, depending on whether they are homogenous as in deep cuts or full of small root holes as in locations nearer the surface.

This field condition obviously will affect the drainage properties of the soil.

Fills consisting of Group A–4 materials settle appreciably, but they come to rest within less than a year. The fills should be placed during the dry season, otherwise they are apt to remain mushy for a long period. This is particularly true for the feebly plastic varieties. Subsequent saturation of a fill by continuous rain also decreases its stability.

GROUP A–5 SOILS CHARACTERIZED BY POROSITY, DEFORMATION, AND REBOUND

Similar to those of Group A–4, these subgrades also consist primarily of very fine sands or silts. But in addition, they contain an appreciable percentage of micaceous particles or diatoms, which cause the subgrades of this group to be highly porous, to deform quickly under load and to rebound appreciably upon removal of load.

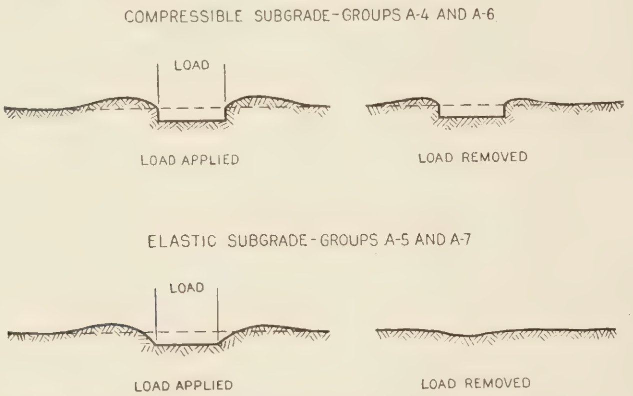
COMPRESSIBLE SUBGRADE-GROUPS A-4 AND A-6
LOAD
LOAD APPLIED
LOAD REMOVED
ELASTIC SUBGRADE-GROUPS A-5 AND A-7
LOAD
LOAD APPLIED
LOAD REMOVED

Figure 16.—Effect of Applying and Removing Load on Compressible and Elastic Subgrades

The quick deformation is due, not to any appreciable volume change, but merely to the small resistance of the soil against distortion. The difference in the character of deformations in a compressible (Group A–4) and an elastic (Group A–5) subgrade is illustrated in Figure 16. Instead of compressing, the elastic subgrade under load squeezes out in a jellylike manner from beneath the load.

The difficulty which this elastic property causes when one attempts the construction of macadams is well illustrated by a highly micaceous soil located in North Carolina. According to C. N. Conner, efforts extending over a period of three weeks failed to effect adequate compaction of the stone courses in one instance and a concrete pavement was subsequently constructed. The road surface suffering from ruptured bond shown in Figure 3, rests upon a subgrade of this group located in Rock Creek Park, Washington, D. C.

The porous and elastic properties of these subgrades are characteristics of the raw materials irrespective of field conditions.

They are identified by laboratory test results as follows: Plasticity index seldom above 10; shrinkage limit above 30; lower liquid limit and field moisture equivalent relatively high, being not less than 30 for a plasticity index of 0 and abnormally higher for soils with greater plasticity indices.

The highly micaceous soils occur very frequently in Pennsylvania, Maryland, and North and South Carolina and other States. The diatomaceous soils were found, among other States, in Maryland and Virginia.

Soil A of Figure 12–A, classed as a gray silt by the United States Bureau of Soils, is an exceptional example of the diatomaceous subgrades, having a shrinkage limit of 60, plasticity index of 19, lower liquid limit of 81 and a field moisture equivalent of 87. When dry this soil weighs but 55 pounds per cubic foot. Adding only 20 per cent of mica flakes to a friable soil may reduce its weight from about 110 to 85 pounds per cubic foot. The expansive properties of sand-mica mixtures have been disclosed in a previously published report (*19*).

Presence of free water and frost action are the two field conditions which influence the performance of these subgrades. Their effect is similar to that exerted on the performance of the Group A–4 subgrades.

GROUP A–6 SOILS, CONSISTING OF CLAY, GREATLY AFFECTED BY FIELD CONDITIONS

These are the subgrades generally termed clay. When dry they take up water in appreciable amounts and suffer proportionate expansion. However, water penetrates the voids at a very much slower rate than it penetrates those of the Group A–4 subgrades. On account of their very appreciable cohesion these subgrades when in the plastic state do not lose their stability, unless they are manipulated as, for instance, by repeated movements of superimposed pavements.

The effect of manipulation (traffic) on the cohesion of these soils is the result of the breaking up of the minute details of the structure. The importance of this loss of cohesion was clearly disclosed by laboratory tests (*20*). Similar effects of pounding on cohesion were also observed with other materials. Thus, by merely hammering the surface of plasticine (a very plastic mixture of fine dust and lubricant), the ultimate bearing capacity of this material is reduced by at least 50 per cent.

All of the Group A–6 subgrades have the following properties in common: Clay content not less than 30 per cent, more than 60 per cent of which consists of fine particles not larger than 0.002 millimeter in diameter; effective size less than 0.002 millimeter; plasticity index not less than 10; shrinkage limit never more than 30 and generally less than 20; field moisture equivalent seldom above 35; centrifuge moisture equivalent seldom below 30 and tendency of specimens to water-log when centrifuge moisture equivalent exceeds 50. Therefore, the following laboratory tests can be used for identifying the members of this group: Wet mechanical analysis, Atterberg plasticity tests, the centrifuge and the field moisture equivalent tests.

These subgrades are very common. As typical representatives we may mention the Mississippi gumbo, the Louisiana gumbo, and the zone B of the Leonardtown series (soil J, fig. 12–C.)

Among the field conditions which are apt to influence the quality and the performance of the Group A–6 subgrades, consisting of identical soil materials, the following should receive special attention: First, whether the soil as encountered in the subgrade is homogeneous (putty-like appearance, without any visible cracks, root holes, etc.) or whether it is full of cracks, root holes, and other visible openings in which the water can freely circulate. The first kind of clay can be considered practically impermeable, incapable of containing anything like free ground water, while the second is more or less permeable and capable of containing ground water.

In order to illustrate the tremendous difference which exists between these two natural states, the two following examples may serve:

In 1927, test borings were made in a typical Mississippi gumbo at the Arkansas approach to the Harahan Bridge at Memphis, Tenn. The clay had a plasticity index of about 40 and a liquid limit of about 75. (Note difference between these values and the plasticity index of 19 with a lower liquid limit of 81, previously given for a Group A-5 subgrade.) In a homogeneous state (worked with water into a homogeneous, plastic paste) it was found to be practically impermeable. It is known to exist in this state in certain parts of Louisiana. However, at the site of the borings, down to a depth of at least 8 feet, the clay was found to be full of root holes in which the water could freely circulate. When withdrawing the drilling tools one could hear the water spouting out of the holes.

In 1928, in Spanish Honduras, Terzaghi encountered a very similar material. Within the forests, in a perfectly undisturbed state, down to a depth of at least 12 feet, the clay was full of root holes and contained free ground water. Permeability tests performed on perfectly undisturbed samples showed that the coefficient of permeability averaged 0.015 centimeter per minute. Then the texture of the material was completely destroyed, thus securing perfectly homogeneous samples consisting of exactly the same material and exactly the same water content as before. After this operation the coefficient of permeability was found to be equal to 0.0000002 centimeter per minute. The undisturbed material, therefore, showed a permeability seventy-five thousand times greater than the disturbed sample.

In nature the same clay may occur either in the first or in the second state. Depending on this state it can either be readily drained or not at all which, as a matter of course, has a tremendous influence on the performance of the subgrade.

Clays in a perfectly homogeneous state are encountered almost exclusively in fairly deep cuts. In this state, exactly the same clay may either be stiff, medium, or soft, depending on its water content. Hence the water content determines whether the supporting quality is good, medium or poor. Since the clay in a homogeneous state does not contain free ground water and since, in addition, the capillary movement of the water is exceedingly slow, there is very little or no frost heave to be expected. On the other hand, the pavement may suffer badly from unequal shrinkage of the subgrade. For the same clay this effect will be the greater, the greater the original water content. For different clays of equal consistency (compressive strength per unit of area of an unconfined specimen) the clay with the higher plasticity and higher liquid limit will be the worse.

The behavior of the porous varieties of the Group A-6 clays will essentially depend on the degree of permeability and on the position of the highest ground-water level. If the ground-water level is permanently low, due to favorable drainage conditions, the subgrade is stable and there is little frost heave. Under the influence of traffic, the subgrade becomes more and more compacted. On the other hand, if the ground water level is high, there exists the danger of very appreciable frost heave and the supporting quality of the soil is likely to be poor.

Thus the interaction between the Group A-6 subgrade and the pavement may be very different, depending on whether the clay is encountered in a homogeneous or in a porous state. Therefore one clay may be very poor in cut and good in natural ground surface, another very good in a cut and very poor in natural ground surface.

If used in fills, the soils of the Group A-6 subgrades are apt to settle during a period exceeding one year. Both the quality of the fill and the speed of consolidation will depend on whether the clay readily slakes and sloughs or whether it is loath to take in water. In this respect, clays with a high plasticity and a high liquid limit (poor in cut, due to great unequal shrinkings) may be more favorable than rapidly slaking clays with a low plasticity index and a low liquid limit (good in cut). The same clay, extracted from the same cut, may furnish a good or a poor fill, depending on whether it was placed during the dry or the rainy season.

Figure 17 illustrates one of the many ways in which field conditions may influence Group A-6 subgrades.

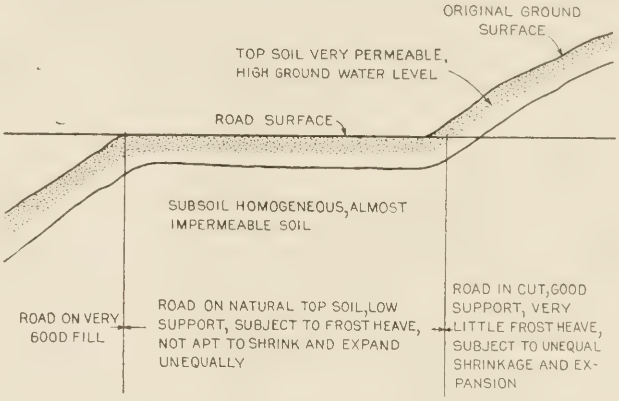
ORIGINAL GROUND SURFACE
TOP SOIL VERY PERMEABLE, HIGH GROUND WATER LEVEL
ROAD SURFACE
SUBSOIL HOMOGENEOUS, ALMOST IMPERMEABLE SOIL
ROAD ON VERY GOOD FILL
ROAD ON NATURAL TOP SOIL, LOW SUPPORT, SUBJECT TO FROST HEAVE, NOT APT TO SHRINK AND EXPAND UNEQUALLY
ROAD IN CUT, GOOD SUPPORT, VERY LITTLE FROST HEAVE, SUBJECT TO UNEQUAL SHRINKAGE AND EXPANSION

FIGURE 17.—CROSS SECTION SHOWING EFFECT OF FIELD CONDITIONS ON THE QUALITY OF GROUP A-6 SUBGRADE

GROUP A-7 INCLUDES CLAY SOILS WITH ELASTIC AND HIGHLY EXPANSIVE PROPERTIES

Similar to the subgrades of Group A-6, those of this group may contain a high percentage of clay. Differing from the subgrades of the foregoing group, those of this one may either contain only a very small percentage of finer clay particles or they may contain an abundance of such material combined with organic matter or coarse-grained mica in the soil. In both cases the voids will average very much larger than those of the Group A-6 subgrades. This in turn causes the Group A-7 subgrades to have higher permeability and smaller resistance to the flow of water toward the surface of evaporation than the A-6 subgrades. Consequently these subgrades behave differently, both when exposed to free moisture and when exposed to the air.

When exposed to free moisture, the A-7 subgrades take in water more rapidly than those of Group A-6. Thus when resting under water for the same period of time, the depth of softening in the A-7 subgrades will equal many times that in the A-6 subgrades.

To illustrate the difference in the two subgrades when drying, let a large ball of each having equal consistency initially, dry under similar conditions for the same period of time. Upon examination, the ball of A-6 material will be found to have a soft core surrounded by a dry crust. The moisture content of the A-7 material in contrast will decrease uniformly from the center to the surface of the ball.

These examples explain why wetting and drying under field conditions produce in A-7 subgrades

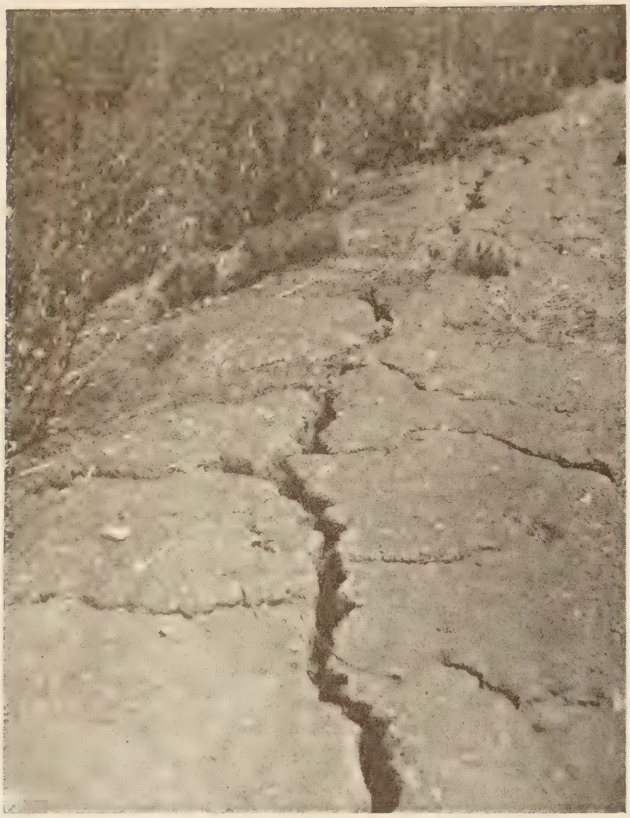

Figure 18.—Shrinkage Cracks in Heavy Clay Soil

effects more detrimental than those produced by similar field conditions in A–6 subgrades. Only when moist are these subgrades spongy or jelly-like. Thus they differ from the subgrades of Group A–5 which have elastic properties also when dry.

The Group A–7 subgrades show laboratory test results similar to those of Group A–6 except as follows: Effective size greater than 0.002 millimeter except when soil particles are grouped due to coagulation (when so grouped the hydrometer test indicates effective size as being greater than 0.002 millimeter); shrinkage limit less than 30; field moisture equivalent may exceed 35; for values above 30, field moisture equivalents more nearly equal to the lower liquid limit than in subgrades of Group A–6 and may exceed centrifuge moisture equivalent; nonwater logging in centrifuge moisture equivalent test.

The Illinois gumbo is representative of the Group 7 soils. In one instance during the construction of the Bates Road, this soil, when wetted by a slight rain, expanded from a compacted semidry state and raised the elevation of the subgrade surface ½ to ¾ inch over night. This soil, when it contained about 30 per cent of water and when deformed in amounts varying between 0.02 and as much as 0.29 of an inch, was almost perfectly elastic. Within these limits the deflections increased consistently at the rate of 0.01 of an inch for each additional pound per square inch soil pressure (*2*). The elastic properties of the Illinois gumbo were very clearly disclosed by repeated application and removal of loads in the field (*3*). The soils of Kansas and Missouri which caused irregular cracks in concrete pavements as previously noted also are representative of the soils of this class.

The inefficiency of macadams when laid on elastic soils of this group is demonstrated by the results of the Bates Road test. Pressure-cell observations indicate that the stability which the macadam bases received during construction was very much reduced by the application of the lightest wheel loads (2,500 pounds). Failure of these bases due to lateral flow of the subgrade appeared soon afterwards (*3*).

The field conditions are apt to exert an influence on the quality of the Group A–7 subgrades similar to that described for the A–6 subgrades. Long wet spells followed by long periods of dry weather may produce very detrimental volume change as shown in Figure 18. With well-distributed rainfall and fairly constant position of the ground-water level these subgrades furnish good support for concrete pavements provided there is no excessive frost action. Because of their elasticity the A–7 subgrades seem less apt to become compacted under traffic than do the permeable varieties of Group A–6. The rapid expansion of the A–7 subgrades which causes the occurrence of irregular cracks in concrete before setting, occurs only when these subgrades contain a particular moisture content during the placing of the concrete. This property is therefore dependent on field conditions.

GROUP A–8 SOILS INCLUDE PEAT, MUCK, AND SIMILAR MATERIALS

Subgrades consisting of peat, muck, and similar materials which afford exceptionally low support are easily recognized in the field without test. Their properties were described in a paper published by V. R. Burton (*21*).

NONUNIFORM SUBGRADES DISCUSSED

Group B–1.—These sugbrade conditions can be due to three different causes; viz, fairly rapid succession of soil types within the top layers of the ground, fairly rapid change in the field conditions (soft and stiff consistency, dense or loose texture, etc.), or a fairly rapid change in the soil profile.

Typical representatives of the first cause may be found in those cases where the subgrade consists of clay with pockets or streaks of sand or where the subgrade consists of sands with layers or pockets of clay. The effect of field conditions on the quality of the subgrade will be approximately as important as their effect on the quality of soil type which dominates. (See effect of field conditions on uniform subgrades A–1 to A–7.)

The second cause of lack of uniformity is illustrated by stiff, Group A–6 clays with pockets of the same clays of softer consistency. It leads to local variations in the quality of support or in the effect of wetting and drying.

The third cause indirectly leads to lack of uniformity by reason of unequal field conditions. An instructive example was presented in Figure 12–C. Due to the presence on the right side of the profile of an impermeable bottom stratum at a moderate depth below the surface, the silt above remained permanently saturated and the pavement broke up. On the left side the impermeable bottom stratum was located at a great depth. As a consequence, the top layer was well drained, and the pavement remained in good condition.

Group B–2.—These subgrades are obtained by constructing fills with soils having very different physical characteristics. Fill material obtained from ground stratified in a horizontal direction is automatically mixed by the steam shovel and the lack of uniformity in the fill will be negligible. On the other hand, if the steam shovel works in succession in materials of different character, the fill will be very nonuniform and the road surface will suffer from unequal subsidence and unequal supporting power of the foundation.

Group B–3.—In passing from cut to natural ground surface, the road passes over the same soil existing under very different field conditions. The effect of this difference in field conditions will be the more important as the character of the soil approaches the character of a highly plastic clay. For nonplastic soils difference between cut and natural ground is insignificant. If the subgrade encountered at the bottom of the cut is in itself nonuniform, we face a combination of Groups B–1 and B–3.

Similar conditions exist if a road passes from cut or natural ground surface into fill. Since fills inevitably settle, while the natural ground does not, the boundary between the fill and the cut inevitably presents a weak zone. For equal fill materials, the pavement defects will be the more important, when there is lack of compaction during construction.

Well compacted mixtures of stones, grit, and dust or clay (residual top soils) as a rule make good side hill fills. There are, however, exceptions to this rule. Thus the residual soils derived from Virginia and Kentucky carboniferous shales may be perfectly stable or flow out, depending on whether they are located on dry ground or whether they rest above fissures or seams through which water percolates out of the ground into the fill. The same is true for side-hill fills of clay.

From what precedes, it is apparent that the results of laboratory tests are sufficient for deciding to which one of the Groups A–1 to A–8 a subgrade belongs. This identification depends, however, not so much upon the results furnished by any particular test as upon the interrelation of results furnished by different tests. It should also be emphasized that the identification can be disclosed by different test combinations. The principal goal of future research in this line merely consists in finding out which particular test combination makes it possible to obtain the desired result at a minimum expenditure of time and money. On the other hand, it also becomes clear that within each group, particularly the groups A–6 and A–7, two subgrades with identical raw material can behave very differently, depending on the field conditions. Hence, in order to evaluate the quality of a subgrade, both the results of the laboratory tests and the results of the field survey must be given equally full consideration.

BEARING PROPERTIES DEPEND ON THE COMBINED EFFECT OF COHESION AND INTERNAL FRICTION

In the preceding sections of this paper, the difference in bearing power, or the capacity of soils to resist the penetration of loads, has been repeatedly presented as one of the most outstanding and distinguishing features of the different subgrades. In the following discussion there is presented an analysis of the physical factors upon which the ultimate bearing capacity of a soil depends.

It is almost universally recognized nowadays that the ultimate resistance of soils to the penetration of heavy bodies depends on the following two physical properties of the loaded soil: (*a*) The cohesion (that part of the resistance to shear which is independent of the outside pressure that acts on the soil); and (*b*) the internal friction (that part which increases in direct proportion with the pressure). The internal friction is normally expressed by the angle whose tangent is equal to the ratio between the frictional resistance and the pressure which causes it (angle of internal friction).

It should be understood that cohesion and internal friction in every respect correspond to the shearing strength of solid bodies. They represent the combined result of the properties of the soil constituents and of the conditions under which the soil exists. Thus they include friction and true adhesion between the individual particles, skin friction between soil particles due to capillary pressures, etc. When these conditions change, both the cohesion and the internal friction of the same material are apt to change, similar to the change in shearing strength of a solid body due to change in the temperature. Therefore, strictly speaking, the terms cohesion and internal friction refer not only to definite materials but also to definite states. In general, the finer the soil constituents the more variable both quantities become.

Since most of the theories of bearing capacity are somewhat elaborate, a crude, approximate method is presented for computing the influence on bearing capacity of internal friction, cohesion, and the other factors on which the bearing power depends. The purpose of the analysis is merely to demonstrate the relative importance of the various factors involved. It may be noted that the results furnished by this approximate method are, numerically, not very different from the results furnished by the more refined analyses, and qualitatively there is no difference at all.

In connection with all the following computations it is assumed that the load acts directly on the surface of the natural ground.

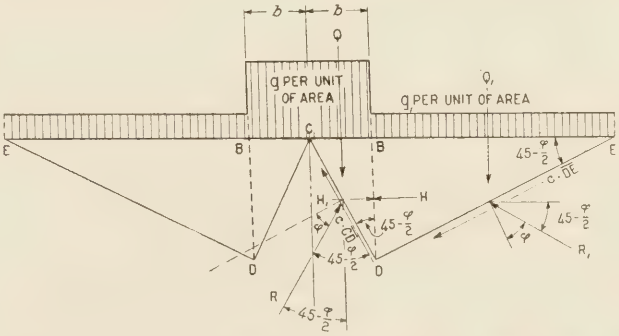

FIGURE 19.—DIAGRAM USED IN COMPUTING THE INFLUENCE OF INTERNAL FRICTION AND COHESION ON BEARING POWER

The principle of the method is represented by Figure 19. Similar methods were repeatedly used by other investigators. (See, for instance, H. Krey, Erddruck und Erdwiderstand, Berlin, 1926.) The strip, loaded with q per unit of surface, has a width of $2b$ and is supposed to be very long as compared with its width. The remainder of the surface of the ground is loaded with q_1 per unit of area. The cohesion of the soil is assumed to be equal to c per unit of area, and φ denotes the angle of internal friction. When the strip is loaded the prisms CBD tend to slide downward. But, before they can do this, they must force the prisms BDE in an upward direction. The prism CBD exerts against BD the "active earth pressure" H_1, and the sliding plane CD forms with the vertical an angle of $45° - \frac{\varphi}{2}$, as does the sliding plane in the back fill of a vertical, perfectly smooth retaining wall. On the other hand, the force required to move the soil BDE out and upward requires overcoming the passive earth pressure H, and, according to the laws of the earth pressure theory, the sliding plane DE forms with the horizontal an angle of $45° - \frac{\varphi}{2}$. Along the sliding planes CD and DE there acts first of all the friction (tangential component of R and R_1, respectively) and then the cohesion (c) per unit of area. Cohesion and friction acting along the inner surface DB are neglected. From the condition, that the forces Q, R, and H acting on CBD, and the forces Q_1, R_1, and H_1, acting on BDE must be in equilibrium with each other, we obtain the following equations:

$$H = Q \tan\left(45° - \frac{\varphi}{2}\right) - 2bc$$

and

$$H_1 = \frac{Q_1}{\tan\left(45° - \frac{\varphi}{2}\right)} + \frac{2bc}{\tan^2\left(45° - \frac{\varphi}{2}\right)}$$

The vertical force Q consists of the external load qb and the weight of the prism BCD, which is $\frac{b^2 s}{2 \tan\left(45° - \frac{\varphi}{2}\right)}$ wherein s is the weight per unit of volume of the soil. Hence,

$$Q = qb + \frac{b^2 s}{2 \tan\left(45° - \frac{\varphi}{2}\right)}$$

In a similar way we obtain

$$Q_1 = \frac{q_1 b}{\tan^2\left(45° - \frac{\varphi}{2}\right)} + \frac{b^2 s}{2 \tan^3\left(45° - \frac{\varphi}{2}\right)}$$

Finally, if q represents the ultimate bearing capacity, the two forces H and H_1 should just balance each other. Hence

$$Q \tan\left(45° - \frac{\varphi}{2}\right) - 2bc = \frac{Q_1}{\tan\left(45° - \frac{\varphi}{2}\right)} + \frac{2bc}{\tan^2\left(45° - \frac{\varphi}{2}\right)}$$

By introducing into these terms the values Q and Q_1 and solving for q we obtain

$$q = \frac{q_1}{\tan^4\left(45° - \frac{\varphi}{2}\right)} + \frac{bs}{2 \tan\left(45° - \frac{\varphi}{2}\right)} \left[\frac{1}{\tan^4\left(45° - \frac{\varphi}{2}\right)} - 1\right] + \frac{2c}{\tan\left(45° - \frac{\varphi}{2}\right) \sin^2\left(45° - \frac{\varphi}{2}\right)} \quad \text{- - - -} (1)$$

In deriving this formula the load was assumed to rest directly on the surface of the ground, with no pavement present between the load and the subgrade. Furthermore, the load was assumed to act over a very long strip. Loading the subgrade in this manner will cause lateral flow similar to that, which as shown in Figures 20 and 21, is productive of pavement failure. This assumption, however, does not satisfy the condition produced by a wheel load rolling upon the pavement. Therefore, the analysis based upon this assumption illustrates only the relative influence exerted by cohesion and internal friction upon the stability of soils without claiming to furnish values suitable as a basis for road design.

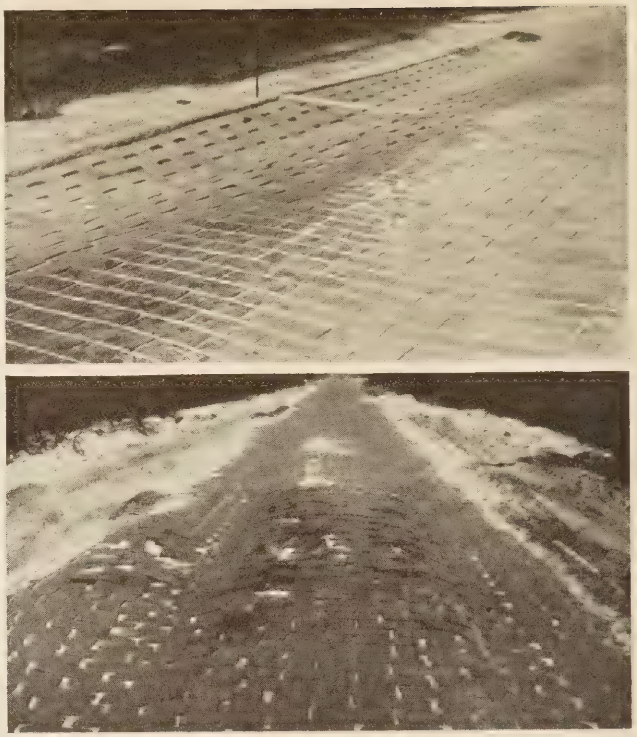

Figure 20.—Lateral Flow of Sand Subgrade Under Brick Pavements in Florida

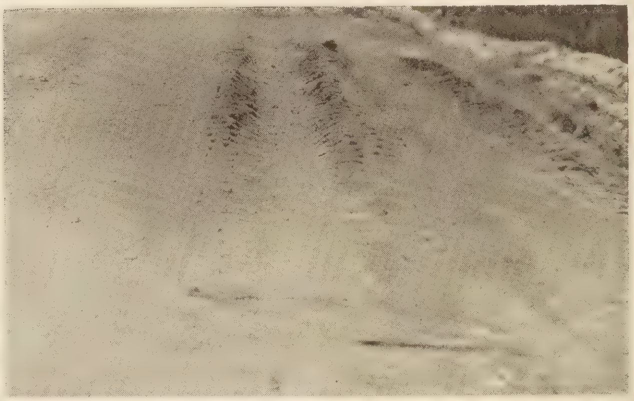

Figure 21.—Failure of Macadam Base on a Group A–7 Subgrade. The Elasticity of the Subgrade Permitted Bond in the Macadam to be Destroyed, After Which Lateral Flow Occurred in the Subgrade

This formula serves to demonstrate the influence of such factors as internal friction and cohesion of the soil, the width of the loaded area and weight applied adjacent to the loaded area upon the ultimate bearing capacity of subgrade soils.

INFLUENCE OF INTERNAL FRICTION AND COHESION ILLUSTRATED

The angle of internal friction which we encounter in practice ranges anywhere between $\varphi = 2°$ for very soft, fat clay of the Groups A–6 and A–7 and $\varphi = 34°$ and more for very well compacted mixed grained sands and for dry silts. (Groups A–1 to A–3 and Groups A–4 and A–5 dry.) The angle of internal friction for stiff clays of Groups A–6 and A–7 and for wet silts Groups A–4 and A–5 is $\varphi = 10°$ (approx). By introducing these values into equation (1) it becomes:

$$q=1.15q_1+0.08bs+4.29c \ldots \varphi=2^\circ \text{ (group A–6 and A–7, soft)}$$
$$q=2.02q_1+0.61bs+5.77c \ldots \varphi=10^\circ \text{ (group A–4 and A–5, wet)}$$
$$q=12.50q_1+10.81bs+17.07c \ldots \varphi=34^\circ \text{ (group A–1 to A–3, stiff)}$$

It should be noticed that the second term on the right-hand side contains weight per unit of volume(s). This fact should call our attention to the effect of a rise in the ground water level on the bearing capacity of cohesionless sands. If the water rises near to the surface of the ground, the specific gravity of the sand apparently decreases on account of the hydrostatic uplift; and, as a consequence, the bearing capacity also decreases. For $q_1=0$ and $c=0$ (free sand surface all around the load, cohesionless sand) the rise of the ground water to the base of the load should theoretically produce a decrease of the bearing capacity by about 35 per cent. This is true since the first and last terms on the right side of the equation become zero and the value of s for dry material and wet material will be in the ratio of 2.65 to 1.65 (2.65 taken as specific gravity of sand which in effect becomes 1.65 when buoyed by water). This is a reduction of about 35 per cent in effective weight and also in bearing value. According to tests performed on a sharp-grained sand with an effective size of 0.183 millimeters and a uniformity coefficient of 2.75 this drop seems to be still more important in practice. Hence the formulas show that the rise of the ground water level to the surface is apt to have a considerable effect on the bearing capacity.

In order to demonstrate the influence exerted by cohesion and internal friction on the ultimate bearing capacity of a soil let it be assumed that $q_1=0$, $b=8.5$ inches or 0.71 foot and $s=100$ pounds per cubic foot.

For a well-graded sand with some mineral binder (Group A–1, traffic-bound) the constants are approximately $\varphi=34^\circ$ and $c=1{,}000$ pounds per square foot. (Binding effect of the fine constituents.)

Hence,

$$q=12.5\times 0+10.81\times 0.71\times 100+17.07\times 1{,}000 = 17{,}838 \text{ pounds per square foot} \tag{3}$$

When the mineral binder required for producing the cohesion c is absent (Group A–3), we have $c=0$, hence

$$q=12.5\times 0+10.81\times 0.71\times 100+17.07\times 0=768 \text{ pounds per square foot} \tag{4}$$

The silt soils (Group A–4 and A–5) when in a semidry condition may have a fairly high supporting value due to high φ and an appreciable cohesion c. When they are thoroughly saturated with water, however, the value of φ may drop to 10° while their cohesion is negligible. Hence their ultimate bearing capacity becomes

$$q=2.02\times 0+0.61\times 0.71\times 100+5.77\times 0 =43 \text{ pounds per square foot} \tag{5}$$

For very soft clay (Group A–6 and A–7, soft) the value of φ is not more than about 2°, but the cohesion is appreciable and can be considered about equal to 200 pounds per square foot. Hence

$$q=1.15\times 0+0.08\times 0.71\times 100+4.29\times 200 =864 \text{ pounds per square foot.} \tag{6}$$

The above figures plainly demonstrate the range of bearing values furnished by varying the values of c and φ in subgrade soils.

INFLUENCE OF WIDTH OF LOADED AREA ON BEARING POWER DISCUSSED

In order to learn about the effect of the width of the loaded area on the ultimate bearing capacity, assume $b=7.1$ feet instead of 0.71 feet as in the preceding example. Results of computations for both widths are shown in Table 3.

For $b=0.71$ feet it will be noted both the bearing capacity of the sand and the soft clay (Group A–3 and A–6) are practically equal (768 against 864) and very small. Therefore, the wheels sink into both materials. However, when the width of the loaded belt is increased to 7.10 feet the bearing capacity of the sand becomes more than 8 times that of the soft clay (7,680 against 915 lbs. per sq. ft.).

Thus, under traffic, clean sand may be almost as unsatisfactory as soft clay, but as a subgrade for a concrete pavement it is far superior to the clay. The more the bearing capacity depends on cohesion, the less significant is the effect of the size of the loaded area on the ultimate bearing capacity.

WEIGHT APPLIED ADJACENT TO LOADED AREA PRODUCES AN EFFECT WHICH IS DEPENDENT ON CHARACTER OF SUBGRADE

To illustrate this influence on ultimate bearing capacity, let a cohesionless sand be confined by a weight, q_1, equal to 100 pounds per square foot. From formula No. 1 the ultimate bearing capacity of the uncompacted sand loaded on an area 0.71 foot wide is, according to computation, equal to 768 pounds per square foot. After application of the confining weight q_1 we obtain

$$q=12.5\times 100+10.81\times 0.71\times 100+17.07\times 0= 2{,}018 \text{ pounds per square foot.} \tag{7}$$

Hence by the application of $q_1=100$ pounds per square foot around the loaded area, the bearing capacity of the sand is increased by 1,250 pounds per square foot.

TABLE 3.—*Ultimate bearing capacity as influenced by width of loaded area*

Width of loaded area in feet	Group A–1	Group A–3	Group A–4	Group A–6
	$\varphi=34^\circ$, $c=1{,}000$	$\varphi=34^\circ$, $c=0$	$\varphi=10^\circ$, $c=0$	$\varphi=2^\circ$, $c=200$
	Pounds per square foot	*Pounds per square foot*	*Pounds per square foot*	*Pounds per square foot*
0. 71	17, 838	768	43	864
7. 10	24, 745	7, 680	430	915

In contrast to its effect on the bearing capacity of soils having high value of φ and low values of c (clean sands), the effect of the same value of q_1 in increasing the support of soils having low values of φ and high values of c (plastic clays) is very small. For instance, the bearing capacity of the very soft clay was found to be equal to 864 pounds per square foot; that is, slightly higher than that of the cohesionless sand. By charging the soil adjacent to the lead with $q_1=100$ pounds per square foot, there results

$$q=1.15\times 100+0.08\times 0.71\times 100+4.29\times 200=979 \text{ pounds per square foot} \tag{8}$$

or an increase of but 115 pounds per square foot compared with an increase of 1,250 pounds obtained in the sand.

Thus to furnish similar increases in subgrade support, the value of q_1 must be considerably greater for soils deficient in internal friction than for soils deficient in cohesion.

Table 4 furnishes additional information on the influence of cohesion and internal friction upon the values of q and q_1. It represents a compilation of what, at present, are considered by the authors the most reliable data, derived from different sources. The figures check reasonably well with those published in Merriman's American Civil Engineers Handbook, fourth edition.

In practice the weight q_1 is furnished by the pavement, and its magnitude depends upon the relative load distribution afforded by the different pavements.

TABLE 4.—*Values of c and φ for different soils and their influence upon the value of q and q_1 for several assumed conditions (b=0.71 foot and s=100 pounds per square foot)*

Soil	Cohesion, c, pounds per square foot	Angle of internal friction, φ	Supporting value, q, pounds per square foot		Increase in supporting value by increasing q_1 from 0 to 100 pounds per square foot
			$q_1=0$	$q_1=100$	
		Degrees			*Per cent*
Clay, almost liquid	100	0	400	500	25
Clay, very soft	200	2	864	978	13
Clay, soft	400	4	1,857	1,991	7
Clay, fairly stiff	1,000	6	4,982	5,134	3
Clay, stiff	1,500	8	8,050	8,225	2
Clay, very stiff	2,000	12	12,528	12,760	2
Silts, wet [1]	0	10	43	245	469
Sands, dry	0	34	768	2,018	163
Sand predominating with some clay	400	30	6,035	6,935	15
Sand-gravel mixtures, cemented [2]	1,000	34	17,838	19,080	7

[1] In silty soils, the angle of internal friction may vary between 10 and 30 degrees but the cohesion may be almost 0.

[2] In properly graded soils (Class 1), depending upon the extent of their compaction, the angle of internal friction may exceed 34° and the cohesion may be considerably less than 1,000.

The preceding computations show that relative stability, with the same binder (mineral or bituminous) depends upon the relative internal friction of the mineral aggregates; and with the same mineral aggregate, relative stability depends upon the relative cohesion of the binders.

Finally, attention should be called to the fact that both internal friction and cohesion of a subgrade can considerably change under the effect of continued traffic. This is true for the subgrades of both traffic-bound and macadam roads. The causes of these changes may be twofold; either mere compacting by vibration, or compacting combined with migration of coarse material into finer substrata, whereby the coefficient of internal friction increases.

PRECAUTIONARY MEASURES FOR OVERCOMING EFFECTS OF UNDESIRABLE SUBGRADE SUPPORT

Measures suggested for preventing the occurrence of undesirable conditions in pavements may be grouped into five classes, as follows:

1. Drainage.
2. Subgrade surface treatments.
3. Subgrade treatments.
4. Base courses and load distributors.
5. Subdivision of concrete pavements and steel reinforcement.

Lack of conclusive information on the efficiency of the methods suggested for improving subgrade support causes part of the following discussion to be more or less theoretical in nature. Therefore the discussion of artificial drainage, subgrade treatments and porous sub-bases suggests proper courses to be followed in experimentation rather than recommendations for established procedure to be used in practice.

DESIGN OF DRAINAGE SYSTEMS SHOULD BE GOVERNED BY KNOWLEDGE OF SOIL CHARACTERISTICS

Saturation of a soil which previously was moist, is apt to cause either a considerable decrease of cohesion (Groups A–6 and A–7) or a decrease of both cohesion and friction (Groups A–4 and A–5). Hence, preventing saturation, by appropriate drainage, can considerably improve the bearing properties of soils of these types. Unfortunately the ability of soils to draw up and retain moisture against the force of gravity, and the ability of ice crystals during their formation to draw moisture up through the subgrade beneath and thus on thawing to liberate immense volumes of free water on top of the subgrade, limit the efficiency of drainage as to means of providing the required stability in all cases (*22*). In spite of this limiting circumstance, artificial drainage, according to existing evidence, may be used beneficially in many instances. The figures in Table 2 show that concrete slabs of equal thickness furnish much less resistance to impact when laid on wet than when laid on drained subgrades. The support offered by the subgrade, shown in the top picture of Figure 22, was so low before draining that the transportation of construction materials was seriously impeded. After the completion of the trench, the support of the subgrade was increased to an extent which eliminated all difficulty of this kind. A similar use of side trenches might have prevented the failures shown in the two lower pictures of Figure 22.

Drainage systems should be designed with regard to the following facts:

1. Where topography prevents the ground-water level from being lowered to a depth greater than the height of the capillary rise, and where the soil is dense and without cracks, stratification and other openings, (homogeneous varieties of Groups A–6 and A–7) artificial drainage structures may be of little use. Otherwise, tile drains and trenches may be used beneficially wherever the moisture content of the subgrade is influenced by the presence of free water (*22*).

2. In northern climates, freezing at the outlets may occur first and in this case the amount of water which may accumulate and freeze under the pavement depends upon the space furnished by the pores in French drains, subbases, etc., which exist there.

3. Tile drains and trenches, to be effective for completely intercepting free water in porous soil strata underlaid by impervious strata, must of necessity extend to the bottom of the porous layer of soil.

4. Under all conditions, adequate provision should be made to drain water through impervious shoulders. It is especially desirable to prevent the water liberated from the top layers of the subgrade soil by thaw from being trapped by the frozen soil beneath and the frozen shoulders on the sides. Figure 23 A shows trapped water of this kind escaping from beneath the pavement through an excavation made in the shoulder. Figure 23 B shows pools of water which remained on an impervious shoulder seven days after a rain. Figure 23 C shows the effect of trapping water on a fill by means of impervious shoulders in the absence of frost action. Figure 24 shows another fill on which water was trapped, water seeping through the abutment of the bridge shown in the first picture, and the water which, in three minutes, gathered in a hole in the pavement surface laid on this fill. This hole was dug seven days after a rain.

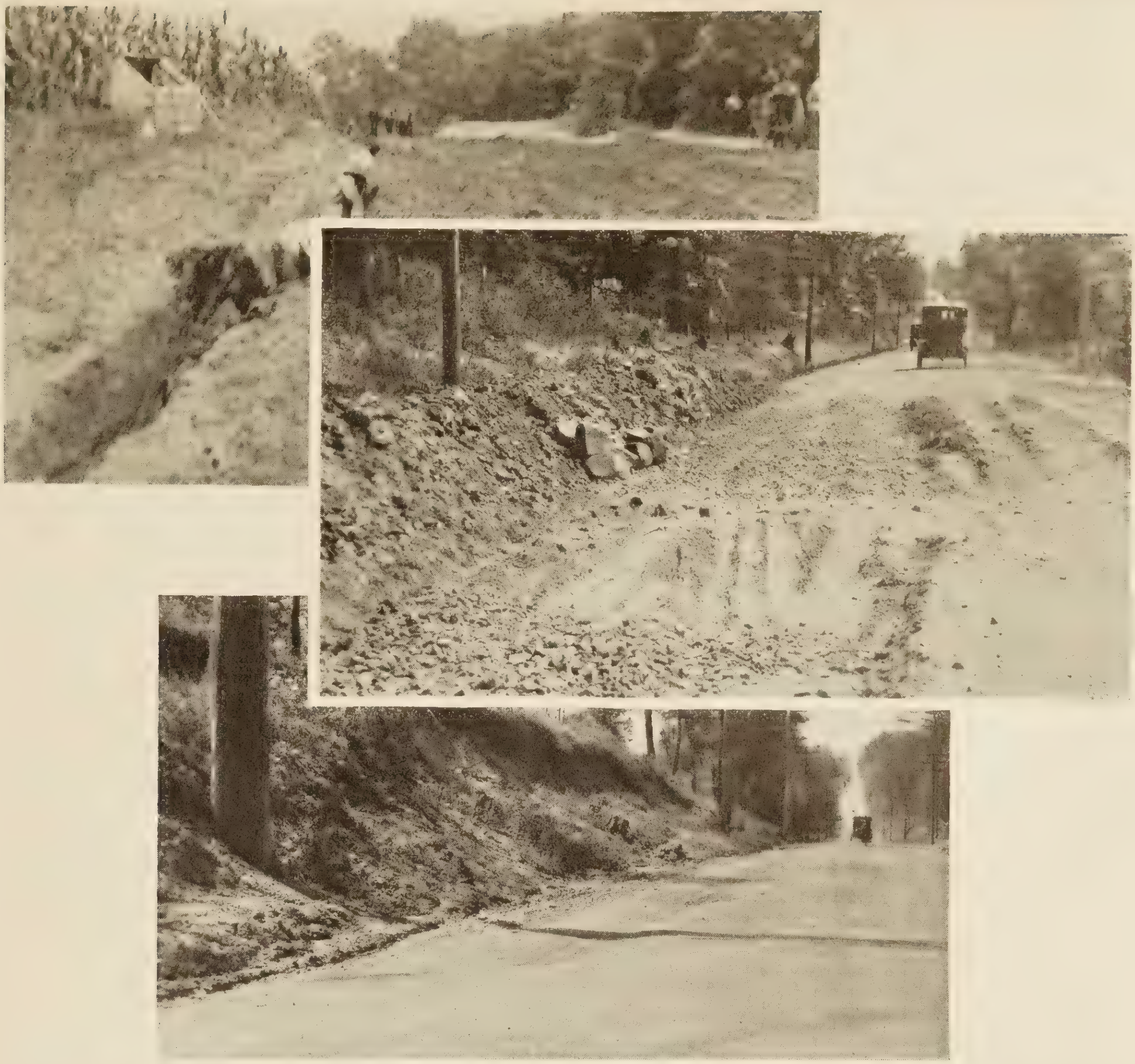

Figure 22.—The Top Picture Shows a Drainage Trench Which Intercepted Seepage Coming From Under the Corn Field at the Left, Thus Increasing the Stability of the Road Surface. The Lower Pictures Show Failures in Cuts Which Might Have Been Prevented by Means of Side Trenches Similar to That Shown Above

Drainage of Group A–1 subgrades merely serves to reduce the danger of frost heave and of Group A–2 subgrades to prevent both frost heave and the softening of the binder.

In the subgrades of Groups A–4 and A–5, water rises by capillarity very rapidly and to great heights. Therefore, to be beneficial in increasing both the cohesion and internal friction and in reducing the danger of frost heave, drainage must keep the ground water level at a very considerable depth below the road surface. Otherwise drainage is ineffective. Special effort should be made to drain fills consisting of Group A–4 and A–5 subgrades.

Groups A–6 and A–7 subgrades can be drained only when they are encountered in a permeable state. When encountered in a homogeneous state (deep cuts) they are practically impermeable and deep side ditches would merely decrease their bearing power and favor unequal shrinkage, without producing drainage. In such cuts it is preferable to confine the drainage measures to the construction of tile drains beneath the shoulders of the concrete pavement, to collect the water which may accumulate between pavement and subgrade.

Fills consisting of Groups A–6 and A–7 subgrades should be as carefully drained as those consisting of A–4 and A–5 subgrades. Fills that consist of very slightly permeable materials with great moisture holding capacity are very loath to give up their water. Hence, they should be placed during the dry season, and measures should be taken to prevent the rain water from penetrating them.

Free water having access to the base of side-hill fills is particularly dangerous (*23*). Free water of this kind when supplied by porous layers located between impervious layers in the supporting soil (*24*) should be intercepted by trenches and tile drain placed along the hill side of the pavement. When the supporting soil throughout is permeable, artificial drainage may not afford benefit and, in this case, the material used for the side-hill fill should be permeable, at least to the same degree.

Free water is detrimental also when it has access to the base of fills located on more or less horizontal ground. When artificial drainage does not furnish a practical means for removing free water which occurs at the base of fills and when relocation of the road is

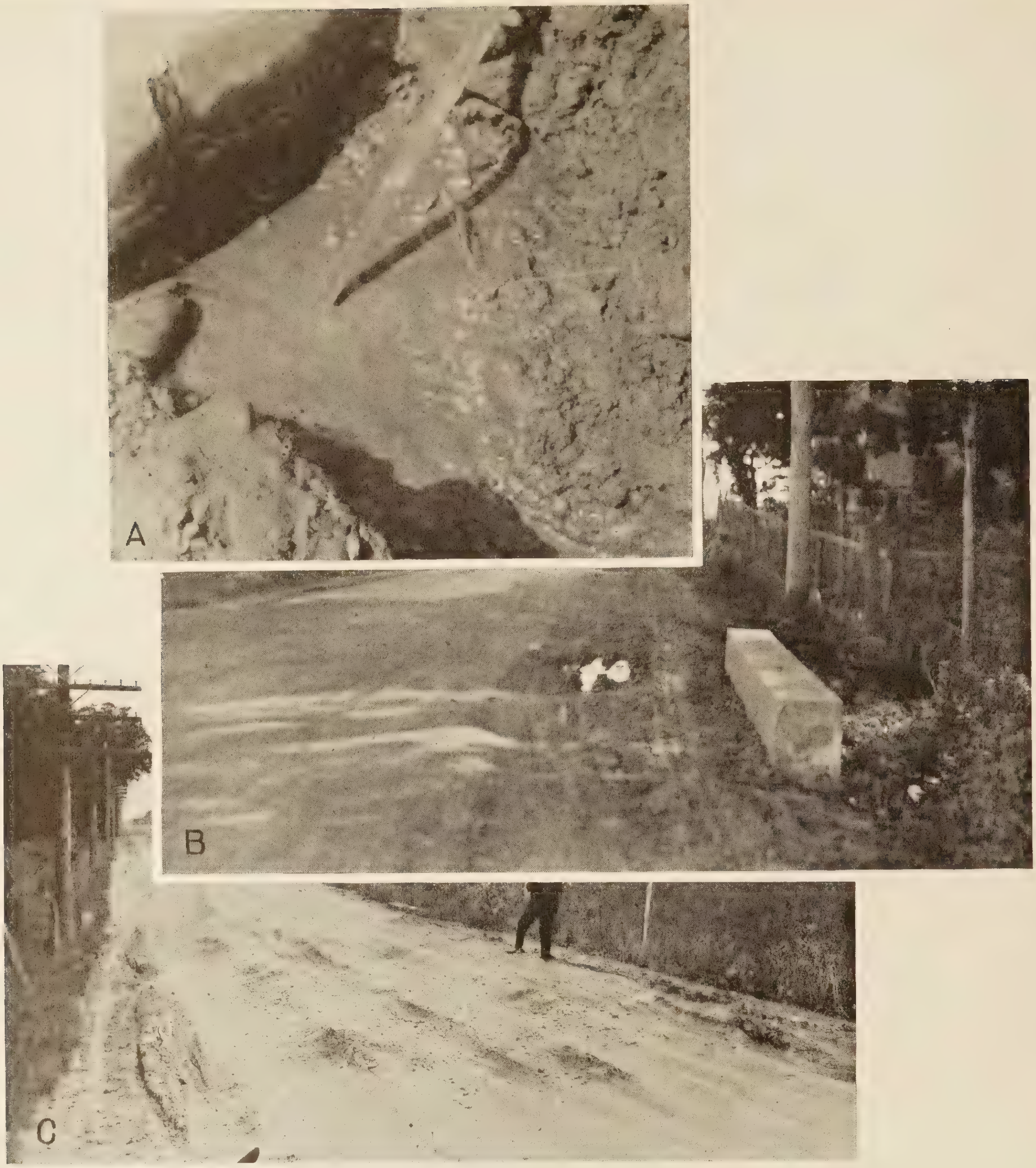

Figure 23.—A, Water Escaping from Beneath Pavement Through Excavation in Shoulder; B, Pool of Water Trapped on Shoulder After a Rain; C, Failure Due to Water Trapped on Fill by Impervious Shoulder

impractical, the fill material must possess stability in an amount sufficient to prevent squeezing out or flowing of the lower portion when wet. Otherwise, pavement failure will occur.

SUBGRADE SURFACE TREATMENTS USED TO PREVENT CHANGES IN CHARACTER OF SUBGRADE

The surface treatments have the common feature that they are confined to the surface or to a very thin top layer of the soil. Their purpose does not consist in changing the character of the top soil but in preventing its change. Therein they differ from the subgrade treatments.

Tar paper covering.—One of the simplest surface treatments for use with concrete pavement consists in covering the surface of the subgrade with a layer of tar paper, prior to placing the concrete, in order to prevent soils of the Group A–7 from detrimental expansion beneath the fresh concrete.

In Iowa, R. W. Crum (*25*) eliminated irregular cracking which occurred in newly constructed slabs by placing a layer of tar paper on the subgrade prior to the laying of the concrete. Whether the cracking referred to was caused by expansion of the subgrade by the water it received from the newly deposited concrete is not known. Tar paper, however, or any other medium which prevents loss of moisture from the newly laid concrete and corresponding increase in the moisture content of the subgrade until after the pavement slabs have developed appreciable strength, may be of benefit for reducing the extent of irregular cracking.

Oiling.—In this treatment the oil merely forms a mat on the surface and does not penetrate the soil appre-

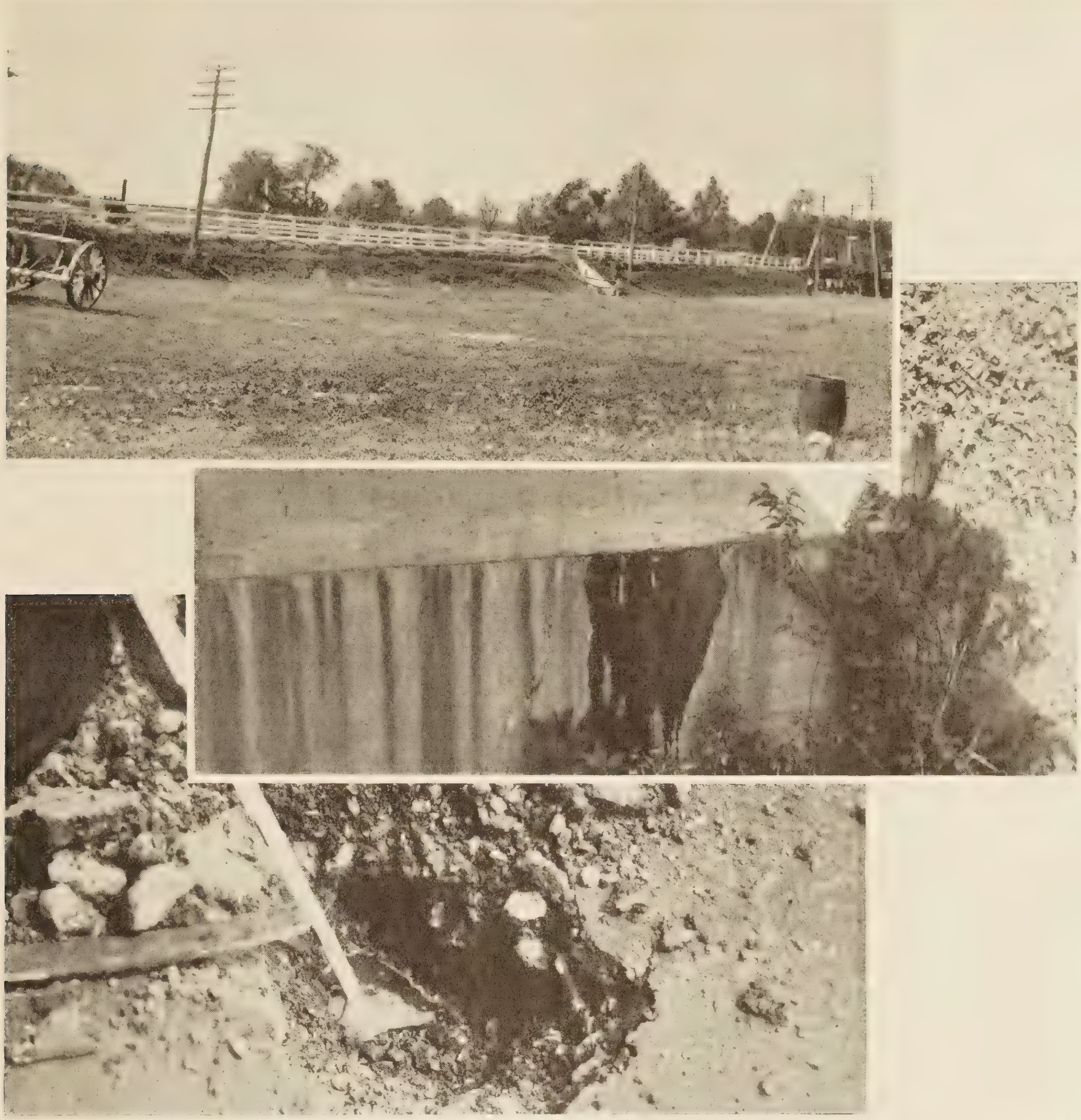

Figure 24.—A Fill in Which Water was Trapped, Water Seeping Through the Abutment Shown in the Fill and Water Seeping in a Hole Cut in the Pavement on the Fill

ciably. This procedure has no effect on the inevitable softening of A–6 and A–7 subgrades as a result of manipulation, but it effectively prevents additional softening which would be caused by moistening from above. Therefore, it reduces the tendency of the soil to work up into the interstices of superimposed layers of gravel, slag, or crushed stone.

Oiling the clay soil before applying a layer of gravel is practiced in Minnesota. This work is in the experimental stage and definite results have not yet been reported. According to F. C. Lang (*26*) this treatment is used on certain gumbo subgrades in which gravel disappears during wet periods.

A treatment of this kind applied to well graded materials quickly determines whether the subgrade belongs to Group A–1 or A–2.

A treatment of this kind tends to prevent evaporation of moisture from the road surface beneath. Therefore, if applied to top soil roads which have not been surface treated previously, it will disclose whether they are comprised of the Group A–1 materials, which do not readily soften due to capillary moisture or the Group A–2 materials, which do. Thus surface treating top soil roads for an appreciable period of time before covering them with a substantial wearing course serves to determine whether the top soil roads in question are apt to furnish the desired support for the proposed wearing course.

Blanket course.—Covering the surface of the subgrade with a thin layer of clean granular material also should reduce the tendency of the soil to work up into the interstices of superimposed pavement layers. However, in this case the purpose is accomplished by the introduction of mechanical obstruction instead of by preventing the top layer of soil from becoming wet. The blanket course serves as a filter which retains the solid part of the clay. For this reason, the blanket course should be fine grained.

On Group A–4 and A–5 subgrades a simple blanket course may prove satisfactory. On Group A–6 and A–7 subgrades the blanket should preferably be combined with oiling.

VARIOUS SUBGRADE TREATMENTS USED TO INCREASE SUPPORTING POWER OF SOIL

The subgrade treatment differs from the surface treatment inasmuch as it has the purpose of changing the supporting quality of the subgrade to a depth sufficient appreciably to change the load distribution over the surface of the natural soil.

This change can be produced by

1. Lowering the moisture capacity of the soil, thus preventing the loss of cohesion due to saturation.
2. Introducing cohesion into cohesionless soils or by increasing cohesion in feebly cohesive soils.
3. Increasing internal friction in cohesive soils.

Admixtures and subgrade manipulation are used in attempts to accomplish these purposes.

Admixtures.—Admixtures which fulfill their purpose of increasing cohesion and friction furnish immense possibilities for increasing the quality of subgrades, provided they are used in stage construction prior to the application of high-grade road surface. The admixtures which have been used experimentally are granular materials, Portland cement, lime, light asphaltic oils, and tars.

Admixtures of the granular materials are proposed to furnish the internal friction necessary for producing stability in the cohesive soils. The water capacity of the mixture of granular material and soil is less than that of the soil alone. This is simply due to the presence of coarse admixture which reduces the percentage of the soil capable of retaining moisture.

Admixtures of the other materials mentioned are proposed merely to reduce the water capacity, thereby reducing the shrinkage and expansive properties, and to retain the natural cohesive properties in soils which, after the incorporation of granular materials, are reduced to mere binders. It is possible also that Portland cement increases the internal friction, and that bituminous materials increase the cohesive properties in fine-grained soils. The bituminous materials in this case must penetrate or be mixed with the soils instead of simply forming a mat on the surface.

Treating the subgrade soil with granular materials, lime, etc., has not received much favor. The results furnished by such treatments have not been positive (*15*) and difficulty has been experienced when attempts were made to mix these materials with the soils (*8*).

Beneficial results may not have been indicated for several reasons. The granular materials may have been mixed with soils which afford poor binding properties; other admixtures may have furnished only additional cohesion and the supporting requirements of the soil may have needed also internal friction; the estimate of the efficiency of the treatment may have been based upon pavement characteristics not influenced by the intensity of subgrade support; or the treatment may not have extended to a depth sufficient to be beneficial.

Applying a bituminous material which penetrates the soil before the granular material is placed upon it, and permitting traffic action to mix the granular material with the treated soil will, until more efficient means are developed, eliminate the mixing difficulty.

The benefits furnished by permitting traffic action to mix the granular material with the soil are indicated by the Ohio experiences referred to previously and also by the behavior of traffic-bound roads in service (*27, 28*). The additional benefit furnished by applying bituminous materials prior to placing the granular material on the subgrade is demonstrated by experience in other States.

For instance, F. V. Reagel found that the quantity of gravel necessary to produce stable road surfaces is considerably less when laid on certain Missouri soils treated previously with light bituminous oils than when laid on soils not so treated.

W. F. Purrington has observed that experimental sections of top soil (fine sand) roads in New Hampshire, when treated with bituminous materials before application of the granular material have exhibited during the winters, stability in amount far exceeding that of adjoining sections not so treated. Similar benefits are obtained in Minnesota by treating sand subgrades with refined tar before applying a thin layer of gravel.

Subgrade treatments affect uniformity merely to the depth to which treatment reaches. Any lack of uniformity which may exist below that level remains unaltered. In a similar way, mixing the subgrade soils for a depth of 12 inches will, according to suggestion (*15*), reduce the undesirable properties in the portion so treated but will not overcome the effects of shrinkage which occur in the soil beneath.

Mechanical manipulation.—The simplest subgrade treatment consists in mechanical manipulation without the addition of other material. The purpose of mechanical manipulation consists either in increasing the density of the subgrade or in producing greater uniformity.

Increase in density may successfully be achieved by the action of traffic. This is apt to consolidate and increase the density of the Group A–4 and A–6 subgrades. Consolidation of the Group A–5 subgrades under traffic can be obtained only in combination with admixtures. If applied to Group A–7 subgrades, manipulation tends to make the top layer more uniform, thus inducing more uniform expansion. However, if applied for the purpose of compensating for lack of uniformity manipulation would be helpful only if this lack were confined to a shallow layer of top soil.

Artificial manipulation for increasing the uniformity may consist only of ploughing, harrowing, and rolling the surface of the sugbrade, as is done in ordinary practice; or it may consist of harrowing, rolling, and wetting the soil in thin layers as was done under one of the sections of the Pittsburg test road (*29*).

BASE COURSES AND LOAD DISTRIBUTORS DISCUSSED

Under this heading porous base courses, compacted base courses, "beam" strength in pavements and thickness in concrete pavements are discussed.

Porous base courses consist of cohesionless materials, such as crushed stone, slag, or gravel without binder. If applied on very fine-grained subgrades they are supposed to:

1. Afford means for draining the subgrade.
2. Reduce the extent of detrimental frost heaving.
3. Replace an equal thickness of undesirable soil.

These alleged advantages are combined with disadvantages as follows:

1. They offer considerable resistance to the transportation of construction materials.
2. They may offer but little resistance to the penetration of the soft underlying clays into the interstices between their particles and consequently may be subject to local settlement and uneveness.
3. They furnish high void volume in which moisture may condense and accumulate when drainage outlets do not function.
4. They may replace an equal thickness of bad material but temporarily.

On the other hand when resting directly upon rock which furnishes free water through a number of crevices, porous base courses furnish an essential part of the drainage system and they should be used, therefore, unless it is convenient to lead the water furnished by the individual veins, away through separate drains.

The question of the drainage benefits to be derived by placing porous base courses on water-bearing subgrades is still in a controversial state and can only be answered by future experimental investigations.

When the thickness of the base course equals the depth to which freezing occurs, substitution of porous material for natural soil will undoubtedly eliminate the effects of detrimental heaving due to frost. There is, however, no conclusive evidence that porous base courses having thickness considerably less than this are beneficial for reducing the extent of frost heave and only future investigation can demonstrate if they cause the heave to be more uniform than that which occurs in the natural soil.

By replacing undersirable soil, porous base courses serve merely to eliminate the softness, lack of uniformity, etc., within the zone which they replace. Therefore they are ineffective for eliminating cracking in concrete pavements caused by frequent change in soil character, or excessive volume change of soil occurring in zones below this depth. Their efficiency for reducing the extent of longitudinal cracking loses significance as a benefit in pavements containing a center joint or groove.

As mere substitutes for undesirable soil, especially when the lack of uniformity is due to unequal volume change, compacted base courses (described below) should prove more beneficial than porous base courses. The low permeability of compacted base courses prevent the penetration of water into and the evaporation of water from the subgrade to a greater extent than the porous base courses.

Porous subbases should always be separated from the Group A–4 to A–7 subgrades, inclusive, by proper subgrade surface treatments and should be adequately provided with drainage outlets.

USE OF COMPACTED BASE COURSE FOR LOAD DISTRIBUTION DEPENDS ON SUBGRADE SUPPORT

Compacted base courses consist of well-graded soils or other materials which possess cohesion and internal friction in amounts sufficient after compaction, to furnish high stability. Thus they include also constructed macadam, slag, or gravel courses.

Compacted base courses serve primarily as foundations for flexible road surfaces although as pointed out above they may serve also to increase the uniformity of support under rigid pavements.

They differ from porous subbases in that:

1. They do not furnish a means for draining water from under the pavement.
2. They provide an excellent riding surface for the transportation of construction materials.
3. They resist the penetration of soft clay from beneath.
4. They furnish a minimum amount of void space in which moisture may condense and accumulate.

Whether it is practicable to use macadam or other nonrigid base courses for load distribution depends entirely upon the subgrade support. The lower this support the greater must be the areas over which the wheel loads must be distributed. And according to previous discussion the load distribution furnished by nonrigid surfaces is small as compared with that furnished by rigid types. Therefore, it may not be practicable to supply the thickness of nonrigid foundations necessary to furnish the load distribution required on subgrades offering very low support. Furthermore, elastic subgrade reaction may interfere with the proper construction of compacted foundations like macadams.

On Group A–1 subgrades, because of their high stability, only a wearing course to prevent abrasion need be constructed. Therefore on highly stable bases, compacted base courses are not needed.

Pavements laid on untreated subgrades of Group A–2 and still more on those of Group A–3, in addition to merely resisting wear, must furnish also the weight necessary to prevent lateral flow in the subgrade. Moderately thick (less than 10 inches) layers of compacted material may furnish the weight required for this purpose.

Thick, compacted foundations, when laid on well drained Group A–4 subgrades and still thicker ones, when laid on the heavy plastic clays of Group A–6, also may render satisfactory service. Treating these subgrades with admixtures and maintaining them under traffic action for a period of time will permit a reduction in the thickness of foundation otherwise required. When this is not practical these subgrades should be surface treated before the foundation is constructed. Compacted foundations should not be constructed on soft Group A–6, poorly drained Group A–3, A–4 nor on Group A–5 or A–7 subgrades until they have been stabilized by stage construction.

Mere thickness in compacted subbases, is not insurance against failure due to penetration of the underlying clays. Heavy telford bases, say 20 inches thick, according to experience, may under these conditions, prove to be no more efficient than light ones.

Depending upon the depth to which the clay becomes liquid, large stones may be swallowed up as quickly as smaller ones. Preventing the clay from becoming soft is the proper remedial measure in this case. Therefore a simple subgrade surface treatment may be much more effective than appreciable increase in thickness, for increasing the service value of nonrigid pavements laid on clay subgrades.

The possibilities of using bituminous binders beneficially in base courses should not be overlooked. Failure of macadam pavements is caused by rupture of the bond existing between the individual stones and the occurrence of this condition, according to theory, is permitted in part only by the low cohesion furnished by the mineral binder. The preventive measure therefore should consist of increasing the cohesion of the binder. Instead of this, a considerable increase in pavement thickness is generally used to compensate for this deficiency in cohesion. The fact that bituminous pavements may resist repeated vertical deflections without distress and that water-bound macadam pavements may be damaged seriously by comparatively small deflections suggests the desirability of performing experiments to determine the efficiency of bituminous binders, when used in base courses, supported by the elastic subgrades of Groups A–5 and A–7.

"BEAM" STRENGTH ESSENTIAL IN PAVEMENTS LAID ON CERTAIN SOIL TYPES

Pavements having high beam strength afford wide load distribution and, therefore, permit but a small intensity of pressure to be exerted on the underlying soil. Thus only a small weight is required to prevent lateral flow. In addition to this, "beam" strength permits the weight furnished by a large area of pavement to be utilized for confining the subgrade soil. Contrasted with this, a pavement having but little "beam" strength permits the load to exert a relatively high intensity of pressure on the subgrade, and thus requires a heavy weight to prevent lateral flow of the soil, and in this case the weight must all be furnished by a small area of pavement. Thus, to accommodate wheel loads of equal weight, pavements without "beam" strength, when laid upon soil offering low support, must be considerably heavier (or thicker) than pavements which have "beam" strength.

"Beam" strength is not essential in pavements laid on subgrades of Groups A–1, A–2, and A–3. It is desirable in pavements laid on Group A–6 and is essential in pavements laid on untreated Groups A–5 and A–7, and poorly drained A–3 and A–4 subgrades.

Thickness in rigid pavements.—Increasing the thickness of rigid pavements increases the factor of safety against the occurrence of "breakage," but not appreciably against the occurrence of primary and secondary traverse cracking.

The figures in Table 2 show the very important influence of slab thickness on the resistance to "breakage.

The results furnished by inspection of a considerable number of concrete roads indicate that the average spacing of transverse cracks in slabs of appreciable age and 6 inches thick is about 18 feet and in slabs 10 inches thick (center) it is about 25 feet. (*7*) This indicates the small influence of slab thickness on the occurrence of primary transverse cracks.

In the Bates Road tests, the wheel load (6,500 pounds) which caused transverse cracks to occur in slabs 6 inches thick and 25 feet long (with center joint) caused them to occur also in slabs of the same length, and 7, 8, and 9 inches thick (*7*). This shows the small influence which slab thickness exerts upon the occurrence of secondary transverse cracks.

A theoretical analysis by Dr. H. M. Westergaard (to be published in the June issue of Public Roads) of the occurrence of cracking in concrete slabs serves to explain these results furnished by surveys of roads in service and by comprehensive tests. According to this analysis, a slab, supported by a uniform elastic subgrade, when cracked by a wheel load will continue to crack until the resultant pieces are not longer than about 5 feet. Increasing the slab thickness of these pieces increases the resistance of slabs to the occurrence of this type of cracking ("breakage").

Also, according to this analysis, cracking in rigid slabs caused when they attempt to adjust themselves to variations in subgrade support in contrast to the foregoing, divides the slab into lengths dependent upon the supporting conditions. Furthermore, increasing the thickness of long slabs in this case, increases instead of diminishes the tendency to crack (primary and secondary transverse cracking). Depending upon the assumed conditions the minimum length of long slabs, according to the analysis referred to, varies between 10 and 18 feet.

The excellent condition, after years of service, of the concrete pavements, 4½ and 5 inches thick, which rested upon California sands; the absence of cracking after 12 years of service in that portion of the Gansevoort-South Glens Falls (N.Y.) road, which rests upon sand and is 4¾ inches thick at the edges and 6¾ inches thick in the center; the good condition after 12 years of service, of the Du Pont Road, Delaware, which rests upon a gravel subgrade and is 5 inches thick at the edges and 7 inches thick in the center; and the excellent service rendered by pavements laid on concrete bases 4 inches thick, which rest on sand in the vicinity of Cleveland, Ohio, indicate that relatively thin concrete pavements, say for instance 7 inches thick at the edges and 5 inches thick in the center, may furnish adequate load capacity when laid on the Group A–2 and Group A–3 subgrades.

Slabs less than 6 inches thick in the center and 8 or 9 inches thick at the edges should not be laid on any of the subgrades except Groups A–1 to A–3. Furthermore, when laid on wet subgrades and when not reinforced, slabs should be at least 1 inch thicker than when laid on similar subgrades well drained.

NEED FOR DIVIDING PAVEMENTS IN SMALL SLABS INCREASES AS UNIFORMITY OF SUBGRADE DECREASES

The use of grooves and joints (*30*, *31*) permits the direction and location of cracks to be controlled. This prevents cracks from meeting joints or side edges of pavement at acute angles and from forming exceptionally short slabs, which conditions facilitate the ultimate destruction of the pavement. Figure 25, C illustrates an uncontrolled longitudinal crack. In Figure 25, A, it may be noted, the corner is broken down where the irregular transverse crack intersects the side edge of the pavement at an actue angle. In the foreground, however, where the transverse crack intersects the side edge at a right angle, the corner cracks have not occurred.

A secondary crack, when uncontrolled, may divide a primary road slab 18 feet long into two slabs having unequal lengths, say, 5 and 13 feet, respectively. A secondary transverse crack, when controlled, however, will divide the primary slab of the same length (in an 18-foot roadway containing center joint or groove) into two uniform slabs 9 feet square.

Except for several pavements which are comprised of precast blocks, slabs 9 feet square are smaller in area than the minimum sizes now used in pavements. To call a road which has cracked into slabs of this size a structural failure, however, is more the result of custom than of fact. The oft-mentioned concrete pavement which has rendered service in Bellefontaine, Ohio, for about 30 years, it will be remembered, is composed of slabs about 5 feet square.

According to the suggestion that the combined length of transverse and longitudinal cracks is related to slab area (*7*), the installation of a center joint or groove should reduce the extent of transverse cracking which would otherwise occur in a full-width slab. Results furnished by pavement surveys now in progress, indicate that this is true.

The necessity for dividing concrete pavements by means of grooves or joints into smaller slabs becomes the more urgent, as the uniformity of the support is decreased.

On A–1 to A–3 subgrades, longitudinal crack control is not essential but it helps to improve the pavement appearance. Examination of concrete roads resting upon sand subgrades in Minnesota indicates that even in the presence of frost action appreciable transverse cracking does not occur in slabs when they are no more than 40 feet long. How much longer than this slabs could be made and still afford complete control of transverse cracking is not known.

On A–4 to A–7 subgrade transverse expansion joints placed at intervals of 60 feet, with 1, 2 or 3 transverse grooves, depending on conditions of subgrade support, placed between them will probably permit the complete control of transverse cracking which occurs to an extent equal to that illustrated in Figure 12, A.

Transverse contraction joints consisting of thin metal strips extending from the bottom of the pavement to within about one-half of an inch of the top, with or without dowels oiled and capped, may be used instead of grooves when transverse expansion joints are spaced far apart or when they are omitted entirely. The use of contraction joints instead of grooves increases the percentage of cross sectional slab area which receives the compressive stresses caused by expansion of the concrete. On B–1 to B–3 subgrades (lacking uniformity) grooves or joints are urgently recommended. Figure 26 illustrates a pavement subdivided by means of grooves and joints.

STEEL REINFORCEMENT RECOMMENDED FOR CERTAIN CONDITIONS

According to published information, steel reinforcement when incorporated in concrete slabs is effective for:

1. Furnishing resistance against the occurrence of "breakage" in slabs laid on wet subgrades.

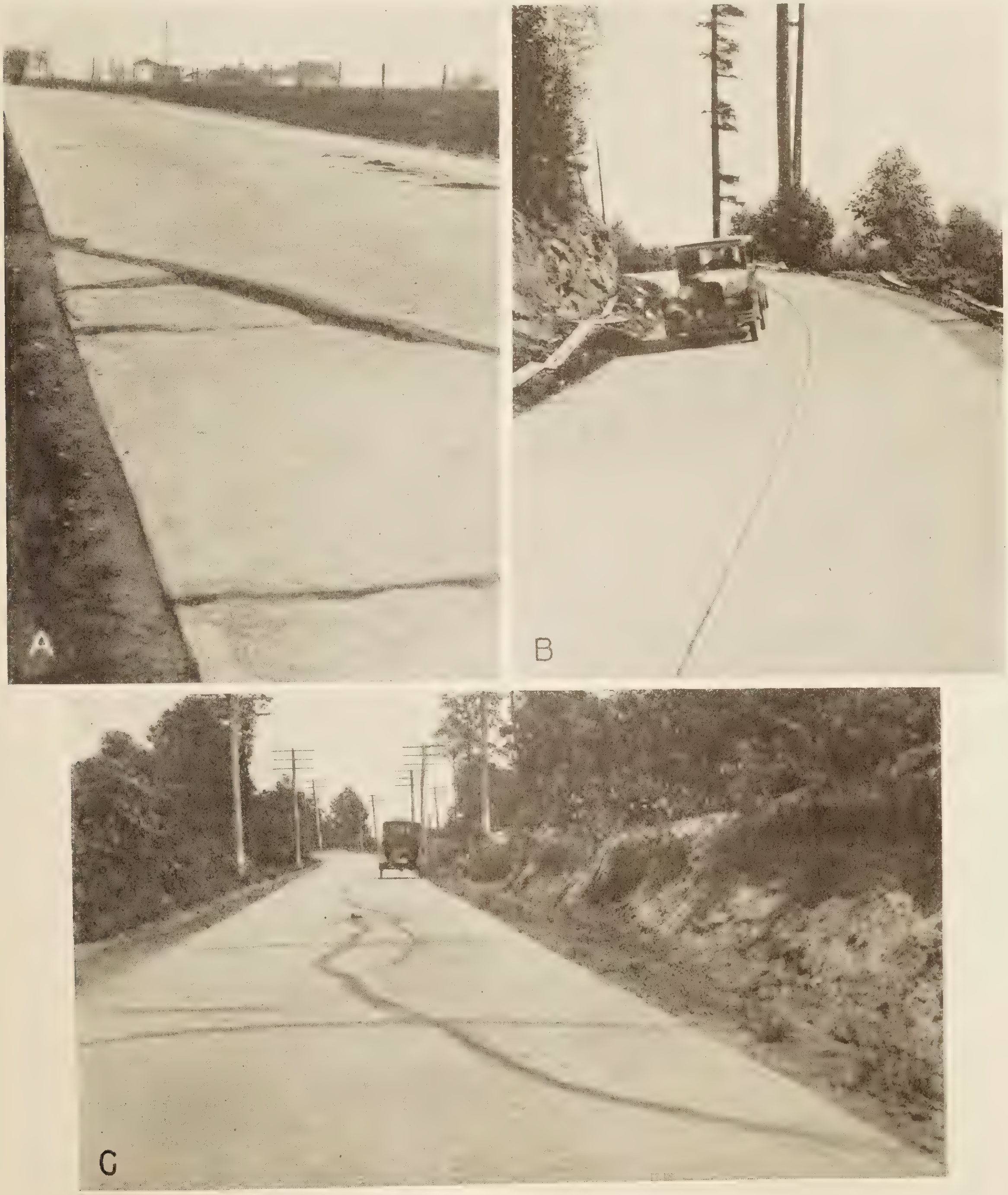

Figure 25.—A, Corner Cracks Occurred Only Where Irregular Transverse Crack Intersects the Side Edge of the Pavement at an Acute Angle; B, National Park Highway in Which Grooves and Joints are Used to Control Longitudinal and Transverse Cracking; C, Longitudinal Crack in Full Width Slab

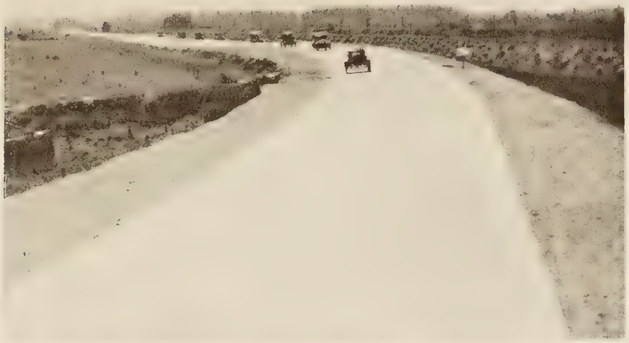

Figure 26.—Concrete Road 20 Feet Wide Divided by Means of Grooves and Joints into Sections 10 Feet by 20 Feet

2. Confining cracks to microscopic dimensions and reducing the extent of visible longitudinal primary transverse, and secondary transverse cracks.

3. Preventing separation in visibly cracked slabs and thereby preventing faulting of slabs, reducing the extent of ravelling along cracks, increasing the smoothness of the riding surface, reducing the magnitude of impacts produced by traffic and increasing the factor of safety against "breakage" of the pavement.

The pavement shown in Figure 23 from beneath which water is escaping, illustrates the beneficial action of steel reinforcement in confining cracks to small dimensions. This pavement contains a center joint and transverse joints spaced 50 feet apart, is laid on a compacted sand and gravel subbase 8 inches thick and is reinforced. When it had been forced upward in amounts varying from 2 to 7 inches, by frost heave during the past winter, fine cracks could be seen in several of the slabs. However, after the pavement had settled to its original position, but few of these cracks were visible. Those which were visible were very fine and did not require maintenance.

Steel reinforcement is recommended for use in pavements laid on wet subgrades of any type, Group A–4 subgrades subjected to frost heave, all Group A–5 subgrades, poor Group A–6 subgrades, all Group A–7 subgrades and on all subgrades in which there exists a very conspicuous lack of uniformity in support (Groups B–1, B–2, and B–3).

SUBGRADE DATA FURNISHES ONLY RATIONAL BASIS FOR PAVEMENT DESIGN

According to the preceding discussion, the selection of type and the design of pavements are not arbitrary matters. Each and every particular pavement variable—stability, "beam" strength, pavement thickness, steel reinforcement, grooves and joints, shoulders, subbases, subgrade treatments, subgrade preparation and artificial drainage has a particular function to perform with regard to the conditions of support furnished by the subgrade. Therefore, subgrade data furnish the only possible basis for rational pavement design. The purpose of subgrade information is not merely to permit the design of pavements having a given load capacity. Instead, its vast economic value lies in the fact that subgrade information furnishes a basis for utilizing the different pavement variables, with respect both to their individual functions and the particular subgrade requirements, in the design of pavements which will transport, at the least cost, wheel loads of specified intensity.

APPLICATION OF PRECAUTIONARY MEASURES TO DIFFERENT TYPES OF SUBGRADES REVIEWED

The application of the precautionary measures to the different types of subgrades can be briefly reviewed in the following manner:

UNIFORM SUBGRADES

Group A–1.—Wearing course sufficient. Drainage to prevent frost heave when ground-water level is high.

Group A–2.—Surface treatment by oiling to prevent softening of binder from above. Drainage to prevent frost heave and softening of binder from below. Load distribution through moderately thick nonrigid or thin rigid courses.

Group A–3.—Coarse materials. Subgrade treatment by admixture of binder or light tars and substantial wearing course. Otherwise moderately thick nonrigid or thin rigid courses. Drainage not required.

Group A–4.—Silts. When naturally drained or when artificial drainage is possible: Thick macadam or concrete pavement of medium thickness (not less than 8–6–8). Subgrade treatment by admixture of coarse constituents permits reducing thickness of macadam. Oiling combined with subgrade treatment may further improve the quality.

When there is a high ground-water level, and drainage is not possible: Macadam unsuitable. Thick concrete pavement (not less than 9–7–9), crack control and reinforcement. Oiling not promising because water comes from below. Subbase may be beneficial for reducing frost effect.

Group A–5.—Same as for group A–4, wet. Condition still more unfavorable.

Group A–6.—Distinguished by the state of the soil, whether impermeable (homogeneous) or permeable (full of cracks and root holes).

Homogeneous state.—Ample load distribution by thick macadam or rigid pavement. Degree of required load distribution depends on degree of softness. Surface treatment (oiling or screenings or both) prevents material from working into nonrigid base course. Crack control for reducing effect of unequal shrinkage.

Permeable state, drainage feasible.—Macadam or rigid type. Subgrade treatment by mechanical manipulation under traffic increases stability.

Permeable state, drainage not feasible.—Very strong macadam or rigid type with crack control. Reinforcement desirable. Subgrade treatment by admixture of coarse material on subbase for reducing frost heave.

In fills.—Mechanical manipulation by traffic very beneficial. Also subgrade treatment by admixture of coarse constituents. Place the fill in dry season. Springs entering the base from below should be piped away. Treatment by oiling may reduce danger of saturation from above.

Group A–7.—Surface treatment by mechanical manipulation for preventing unequal expansion and by application of tar paper for preventing expansion beneath fresh concrete. Otherwise treat them like the soft, homogeneous Group A–6 subgrades.

Group A–8.—Fill on top of soft ground, according to Michigan and Minnesota practice. Pavement requires "beam" strength, ample crack control, and reinforcement.

NONUNIFORM SUBGRADES

Group B–1.—If nonuniform layer is shallow, mechanical manipulation for increasing uniformity. If deep and very nonuniform, rigid pavement with crack control and reinforcement. Subbase may be beneficial.

Group B–2.—Nonuniform fills should be avoided if possible. If not feasible, subgrade treatment by mechanical manipulation, great "beam" strength, crack control, and reinforcement.

Group B–3.—Great "beam" strength, crack control, and reinforcement at boundary between cut and natural ground surface. Fills should be compacted as much as possible before placing the pavement. At least longitudinal crack control. Careful piping away of any springs or water veins which might enter the fill through the base.

CONCLUSIONS PRESENTED

Among the facts on which the preceding discussions were based the following deserve to be emphasized:

SUBGRADE SUPPORT

1. The support furnished by different subgrade soils varies widely with regard to intensity of support (high or low), character of support (firm or elastic), and uniformity of support (constant or variable intensity of support), and thus permits a differential of considerable amount in pavement requirements.
2. The character of support afforded by subgrade soils is dependent upon soil constituents such as sand, clay, silt, mica, and diatoms, and upon the structure of the soil and the field conditions under which it exists.
3. The intensity of support in uniform subgrades depends upon the cohesion and internal friction in the soil, the area of load distribution, and the weight of superimposed pavement.
4. High subgrade support requires either high cohesion or high internal friction, preferably both combined. High cohesion is characteristic of clays with stiff consistency; high internal friction, of well graded sands; and a combination of both internal friction and cohesion of well-graded sands with a binder.
5. The greater the internal friction the more will the support of the subgrade be improved by distributing the load over a larger area or by increasing the dead weight of the pavement.
6. Increasing the intensity of subgrade support permits a reduction in the thickness of nonrigid foundations and increases the factor of safety against "breakage" but not necessarily against longitudinal and transverse cracking in rigid pavements.
7. Eliminating the elastic rebound in subgrades permits the construction of nonrigid pavement types otherwise not suitable.
8. Increasing the uniformity of subgrade support reduces the extent of transverse and longitudinal cracking in rigid pavements and may provide against failure in nonrigid types.

DRAINAGE

9. Side ditches or deep trenches may intercept water furnished by porous strata and may serve to lower the ground-water level. Thus they may be effective for preventing failure in side-hill fills, for increasing the subgrade support, and for reducing the extent of frost heave.
10. Tile laid in trenches filled with porous material and placed under the edges of pavements serve to intercept water entering from the sides and to prevent the accumulation on top of the subgrade. Thus, they may serve the same purposes as deep side trenches and, in addition, reduce the extent of softening of the subgrade due to saturation from the top. Also they provide against the separation of cracked rigid slabs, due to successive freezing of water trapped by impervious shoulders.

SUBGRADE SURFACE TREATMENTS

11. Subgrade surface treatments consisting of oiling and blanket courses serve to prevent the infiltration of soft subgrades into superimposed pavements and thus they may very appreciably increase the service value of nonrigid pavements laid on fine silt or clay subgrades.

SUBGRADE TREATMENTS

12. Subgrade treatments serve primarily to increase the intensity of subgrade support, although, depending upon conditions, they may also increase the uniformity of subgrade support. Thus, they are beneficial primarily only with respect to the construction of nonrigid pavements.
13. Intensity of subgrade support is increased by adding cohesive materials to sands, granular materials to clays, and both cohesive and granular materials to silts. Adding moisture capacity reducers to retain high cohesion, and granular materials to supply internal friction, to clays may increase the benefits furnished by the addition of granular materials alone.
14. Subgrade treatments should be planned on the basis of the information furnished by laboratory tests concerning the soil properties.

BASE COURSES

15. Compacted base courses serve primarily to furnish load distribution in nonrigid pavements.
16. Except when porous base courses extend from the bottom of the pavement to the top of a rock subgrade, compacted base courses may be the more efficient.
17. The penetration of the underlying clay into the voids of porous base courses may furnish conditions of support which, due to their lack of uniformity, may be extremely undesirable.
18. Base courses may reduce the extent of longitudinal cracking which would otherwise occur in rigid pavements. This ceases to be a benefit in pavements which contain a center joint or groove.
19. Except when lack of uniformity in subgrade support is caused by conditions which exist in close proximity to the pavement, the efficiency of base courses for decreasing the extent of primary and secondary transverse cracking in rigid pavements is questionable.

PAVEMENT DESIGN

20. "Beam" strength is desirable in pavements laid on soils whose low support is due to a lack of internal friction.
21. "Beam" strength is not a necessity in pavements laid on soils whose low support is due to the absence of cohesion.
22. "Beam" strength is not an essential requirement for wearing courses on Group A–1 subgrades.
23. Increasing the thickness of rigid pavement furnishes an increase in the factor of safety against the occurrence of "breakage" but not to an appreciable extent against the occurrence of primary and secondary transverse cracking due to lack of uniformity in support.

24. Crack control and steel reinforcement serve to eliminate the undesirable effects caused by such cracking as can not be prevented in rigid pavements. Their use permits a predetermination of the ultimate size of slabs, free from additional visible cracks, and also prevents faulting and separation of the slabs.

25. A subgrade survey record, to be of practical use in the design of highways, should consist of a map showing the ground-water level and the soil profile together with a description of the physical properties (both laboratory and field) of the various soils which comprise the profile.

SUMMARY

26. Each part of the pavement has a particular purpose to fulfill. If it is necessary to have three parts—subgrade treatment, base course, and wearing surface—each is equally important and failure in any one may mean destruction of the pavement. Therefore, selection of the various courses should be made with regard to the duties which they are to perform; and the same diligence and precaution should be exercised in the construction of the lower courses as in the top or wearing course.

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A. S. T. M. SYMPOSIUM ON MINERAL AGGREGATES

The American Society for Testing Materials announces as a portion of the program of the annual meeting to be held at Atlantic City June 25–28, a "Symposium on Mineral Aggregates" which should be of considerable interest to all producers and users of aggregates.

Highway engineers will be particularly interested in papers dealing with Methods of Inspection, by A. S. Rea, chief, bureau of tests, Ohio State Highway Department; Fine Aggregates in Concrete, by H. F. Gonnerman, of the Portland Cement Association; Fine Aggregates in Bituminous Mixtures, by H. W. Skidmore, Chicago Paving Laboratory; Influence of Coarse Aggregate upon the Strength of Concrete, by F. C. Lang, engineer of tests and inspections, Minnesota State Highway Department; Influence of Coarse Aggregate Upon the Durability of Concrete, by F. R. McMillan, of the Portland Cement Association; Effect of Aggregates upon the Stability of Bituminous Mixtures, by Prevost Hubbard, of the Asphalt Association; and Aggregates in Low Cost Road Types, by C. N. Conner, of the American Road Builders Association.

The symposium will be opened by a paper on Organization Problems, by R. W. Crum, director of the highway research board, and chairman of the committee in charge of the program, and will close with a paper on Needed Research in Aggregates, by F. H. Jackson, of the Bureau of Public Roads.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.

DEPARTMENT BULLETINS

No. *136D. Highway Bonds. 20c.
220D. Road Models.
257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370D. The Results of Physical Tests of Road-Building Rock. 15c.
386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387D. Public Road Mileage and Revenues in the Southern States, 1914.
388D. Public Road Mileage and Revenues in the New England States, 1914.
390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407D. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
463D. Earth, Sand-Clay, and Gravel Roads.
*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537D. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660D. Highway Cost Keeping. 10c.
*670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
*691D. Typical Specifications for Bituminous Road Materials. 10c.
*724D. Drainage Methods and Foundations for County Roads. 20c.
1216D. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1259D. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.

DEPARTMENT BULLETINS—Continued

No. 1279D. Rural Highay Mileage, Income, and Expenditures, 1921 and 1922.
1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

No. 94C. T. N. T. as a Blasting Explosive.
331C. Standard Specifications for Corrugated Metal Pipe Culverts.

TECHNICAL BULLETIN

No.55. Highway Bridge Surveys.

MISCELLANEOUS CIRCULARS

No. 62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects.
93M. Direct Production Costs of Broken Stone.
*109M. Federal Legislation and Regulations Relating to the Improvement of Federal-Aid Roads and National-Forest Roads and Trails. 10c.

FARMERS' BULLETIN

No. *338F. Macadam Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

No. 1914Y. Highways and Highway Transportation.
937Y. Miscellaneous Agricultural Statistics.

TRANSPORTATION SURVEY REPORTS

Report of s Survey of Transportation on the State Highway System of Connecticut.
Report of a Survey of Transportation on the State Highway System of Ohio.
Report of a Survey of Transportation on the State Highways of Vermont.
Report of a Survey of Transportation on the State Highways of New Hampshire.
Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio.
Report of a Survey of Transportation on the State Highways of Pennsylvania.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
Vo. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL AID ROAD CONSTRUCTION

AS OF

APRIL 30,1929

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE			Estimated total cost	Federal aid allotted	MILEAGE				
				Initial	Stage[1]	Total			Initial	Stage[1]	Total		
Alabama	1,934.1	$ 3,504,753.15	$ 1,750,350.12	230.4	21.0	251.4	$ 419,161.56	$ 209,580.78	27.4		27.4	$ 2,433,283.96	Alabama
Arizona	881.8	1,619,148.95	1,392,765.11	63.3	29.7	93.0	312,893.67	233,061.00	8.8		8.8	3,162,886.69	Arizona
Arkansas	1,749.9	3,577,132.61	1,773,985.83	99.0	6.5	105.5	807,615.92	403,806.95	20.4		20.4	2,270,660.08	Arkansas
California	1,569.9	10,036,044.98	4,565,587.78	263.7	15.0	278.7	213,030.88	128,755.85	20.1		20.1	2,464,354.71	California
Colorado	1,079.7	3,348,882.81	1,704,561.85	125.1	47.6	172.7	1,175,076.93	607,330.43	31.1	8.3	39.4	2,353,121.96	Colorado
Connecticut	228.9	900,832.42	272,216.34	13.0		13.0	673,564.46	272,075.09	3.6		3.6	844,972.54	Connecticut
Delaware	212.9	757,621.05	300,970.55	15.7		15.7	159,060.00	79,530.00	6.0		6.0	227,939.72	Delaware
Florida	439.5	2,970,573.29	1,253,256.11	100.7	5.4	106.1						1,954,792.48	Florida
Georgia	2,549.2	4,022,399.50	1,800,511.90	178.7	36.9	215.6	27,743.43	13,500.00	2.3		2.3	2,025,783.39	Georgia
Idaho	1,141.5	886,453.40	529,037.61	71.9	3.0	74.9	91,186.67	54,816.95	5.7		5.7	989,611.53	Idaho
Illinois	1,815.5	20,015.962.78	8,930,212.36	603.9		603.9	1,055,946.49	511,771.20	39.3		39.3	2,998,543.72	Illinois
Indiana	1,261.1	6,614,248.73	3,161,443.24	203.2		203.2	3,119,829.61	1,535,520.00	110.3		110.3	304,157.75	Indiana
Iowa	2,988.9	2,761,285.77	1,123,060.58	42.4	89.8	132.2	4,145,323.14	1,839,871.18	58.4	102.3	160.7	52,328.38	Iowa
Kansas	2,456.4	4,934,170.26	1,956,799.09	298.5		298.5	158,463.44	59,960.36	7.5		7.5	2,211,957.38	Kansas
Kentucky	1,309.7	4,624,245.25	2,207,446.31	233.1		233.1	483,624.70	241,812.34	36.5		36.5	861,560.11	Kentucky
Louisiana	1,308.4	4,141,243.94	2,062,863.34	164.9		164.9	103,055.30	25,000.00	.1		.1	1,223,703.42	Louisiana
Maine	480.5	1,779,653.03	596,458.57	40.9		40.9	192,121.39	93,601.55	8.9		8.9	1,449,150.77	Maine
Maryland	627.9	167,810.00	62,360.00	3.6		3.6	228,927.25	107,200.00	4.8	16.8	21.6	563,235.37	Maryland
Massachusetts	568.2	4,909,635.79	1,483,653.33	86.0	.2	86.2	134,046.55	26,130.00	1.7		1.7	1,940,282.10	Massachusetts
Michigan	1,445.7	10,913,955.95	4,648,869.11	261.3		261.3	1,904,320.00	805,886.86	45.8	9.5	55.3	1,696,039.82	Michigan
Minnesota	3,954.8	1,411,778.11	432,618.27	96.5	11.2	107.7						2,065,297.96	Minnesota
Mississippi	1,655.6	4,675,251.55	2,115,579.26	211.9	1.6	213.5	360,016.07	166,712.06	20.9	13.0	33.9	1,360,511.46	Mississippi
Missouri	2,264.7	9,128,276.74	3,444,941.15	204.2	57.3	261.5	4,763,429.34	1,829,431.42	37.5	112.3	149.8	138,067.13	Missouri
Montana	1,531.7	4,250,056.51	2,680,639.21	274.9	7.6	282.5	821,954.50	436,433.17	93.6		93.6	4,723,837.01	Montana
Nebraska	3,552.7	2,902,858.33	1,445,126.61	278.9	77.5	356.4	462,265.14	231,132.53	22.0	57.3	79.3	3,323,183.27	Nebraska
Nevada	1,097.8	838,189.01	729,782.17	64.6	77.4	142.0	153,052.33	135,986.97	14.5	27.3	41.8	866,314.48	Nevada
New Hampshire	331.7	279,253.01	108,613.31	7.5		7.5	139,996.48	41,445.00	2.8		2.8	364,030.54	New Hampshire
New Jersey	454.6	4,846,598.94	826,185.00	55.1		55.1						864,600.08	New Jersey
New Mexico	1,821.6	2,685,815.29	1,696,634.11	184.0		184.0	370,507.18	236,198.30	13.0		13.0	1,081,152.91	New Mexico
New York	2,142.6	22,651,431.43	5,031,100.55	335.9		335.9	5,096,100.33	1,073,355.00	71.7		71.7	5,970,472.09	New York
North Carolina	1,685.4	1,725,523.63	862,761.78	76.0	11.2	87.2	237,676.85	112,358.42	11.0		11.0	1,950,861.83	North Carolina
North Dakota	3,666.8	3,054,041.96	1,278,608.95	452.5	108.6	561.1	1,152,828.67	421,682.78	154.5	207.6	362.1	1,157,118.43	North Dakota
Ohio	1,986.1	11,178,184.19	3,925,153.15	238.0	.1	238.1	3,539,228.79	977,938.29	65.3	9.8	75.1	3,431,416.12	Ohio
Oklahoma	1,748.0	2,652,625.13	1,216,834.00	99.6	25.8	125.4	1,527,951.50	700,462.51	67.4	7.0	74.4	985,134.03	Oklahoma
Oregon	1,147.9	572,651.64	340,807.93	48.1		48.1	80,071.75	49,828.64	6.0		6.0	2,139,914.75	Oregon
Pennsylvania	2,039.0	12,474,771.25	3,449,202.19	209.7		209.7	2,428,401.52	740,498.73	42.3	14.1	56.4	3,078,643.66	Pennsylvania
Rhode Island	165.2	845,570.40	241,756.00	16.1		16.1	363,714.36	84,345.00	5.6		5.6	648,214.23	Rhode Island
South Carolina	1,812.2	4,152,003.50	991,814.40	115.1	37.4	152.5	20,550.07	4,000.00	.1		.1	1,087,856.17	South Carolina
South Dakota	3,298.0	2,710,518.07	1,481,365.34	420.6	34.4	455.0	323,623.75	183,533.55	65.7	13.6	79.3	1,022,117.10	South Dakota
Tennessee	1,112.8	4,354,958.19	1,989,347.00	94.3	38.1	132.4	608,666.35	304,333.17	23.1		23.1	1,225,369.35	Tennessee
Texas	6,078.4	15,005,384.10	6,377,429.15	624.6	179.0	803.6	5,145,117.64	2,303,414.02	144.1	112.2	256.3	2,372,372.28	Texas
Utah	916.9	1,563,640.22	1,037,204.86	67.3		67.3	98,235.67	73,054.39	12.3		12.3	597,844.28	Utah
Vermont	229.0	965,465.68	288,777.15	20.5		20.5	767,771.78	265,706.42	16.9		16.9	150,038.64	Vermont
Virginia	1,335.4	2,035,386.89	858,518.60	48.6	15.2	63.8	919,912.22	436,470.35	47.4	3.7	51.1	969,126.08	Virginia
Washington	841.9	3,870,131.93	1,320,675.25	85.6	18.1	103.7						1,406,300.03	Washington
West Virginia	688.6	1,915,850.26	828,504.13	51.1	12.4	63.5	1,888,388.17	715,388.03	36.9	12.3	49.2	198,544.50	West Virginia
Wisconsin	2,099.4	4,407,486.31	1,929,523.96	133.6	4.9	138.5	2,926,692.78	1,186,335.00	85.2	21.1	106.3	1,951,471.19	Wisconsin
Wyoming	1,693.1	816,626.57	521,608.95	86.5		86.5	353,448.21	204,426.69	31.7	19.1	50.8	789,293.44	Wyoming
Hawaii	39.4	465,809.51	169,701.02	6.7		6.7						1,319,923.77	Hawaii
TOTALS	77,441.0	220,923,192.01	89,197,221.58	7,707.3	972.9	8,680.2	49,954,592.84	20,163,680.98	1,530.2	767.3	2,297.5	79,267,392.69	TOTALS

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.